# WATER POWER

*A technical journal devoted to the study of all aspects of Hydro-electric developments*

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WATER POWER

*A technical journal devoted to the study of all aspects of Hydro-Electric Development*

# Corrosion

A NEW branch of engineering, which is of special interest to those engaged in all aspects of hydro-electric developments, is now emerging—the branch that deals with the prevention of corrosion. The hydro-electric engineer has of course always been intimately concerned with the prevention of corrosion that endangers the efficiency of the turbine runners themselves, and in the study of the factors affecting this aspect of the corrosion problem he has possibly led the engineering world.

Corrosion experts consider corrosion as being of two distinct types and have divided the subject under two headings. The first category is essentially chemical corrosion, in which there is no passage of current from the metal to an external medium such as damp earth, as the anode and cathode zones are intermingled so closely on the metal surface that they cannot be measured. In the case of electrolytic corrosion, however, the assumption is that current flows between the metal itself and some external electrode, through the surrounding medium.

The most typical case of chemical corrosion is the rusting of ferrous metals in air. This has been tackled by means of protective coverings such as galvanising, spraying of hot metal, and painting; while another approach is the substitution of other metals such as bronze, aluminium—or perhaps in the future, titanium—for those metal parts the failure of which might bring serious consequences. In the case of transmission towers and their conductors, the use of stainless steel or aluminium for the upper parts, where maintenance work is hindered by the presence of live conductors, has been tried in a number of countries and has been found to be economic, particularly where the climatic circumstances are such that access to the tower line cannot be gained for long periods.

An interesting paper given before the Institution of Electrical Engineers in January of this year dealt with the results of a 20-year experiment on the corrosion of steel-cored aluminium conductors on the British Grid system. In this it was shown that in industrial areas, where acid fumes may be expected, corrosion took the form of pitting on the outer surface of the top aluminium strands, which gradually worked inwards from the outer layers and then began to attack the inner layers, leaving the steel core intact. In coastal regions, however, where salt air could be expected, the reverse process took place, the corrosion starting as an electro-chemical action between the steel and the aluminium on the inside of the cable, working outwards and causing bulges which eventually broke the aluminium strands. The conclusions reached by this exhaustive survey were that greasing (with very carefully selected types of grease) formed a reasonable economic solution. It was thought that the mean service life, expressed in terms of loss of mechanical strength of the complete conductor, to be expected in Great Britain would be of the order of 25 to 30 years.

Recently, positive methods have become available, and have been successfully tried out in practice, for combating the second category of corrosion, which arises from external electrolytic action. For large steel structures, such as pipelines, sluice gates, and similar structures, cathodic protection appears to offer many advantages. It has been applied of late with considerable success in protecting the hulls of naval ships during periods when the ships are lying in reserve.

Two basic methods are available. In one of these a metal, which is electro-chemically different from that which is to be protected, is "planted" in the ground near the structure, and is connected to it by means of a conductor. The structure acts as the cathode and the new metal as the anode. In this way a battery or electro-chemical cell is created between the expendable metal and the metal to be protected, and this results in the reversal of the electro-chemical action that would otherwise lead to a wasting of the main structure.

In the second method, a small electromotive force, derived from an auxiliary supply, is impressed between the structure and a suitable electrode, which does not necessarily waste away, and which in effect serves the same purpose as in the previous method, by counteracting the natural electro-chemical effect. In the case of hydro-electric stations and their associated pipelines, where ample supplies of auxiliary power are

usually available, this method has much to recommend it, since there is no need for periodical replacement of the anode, and all that is necessary for its maintenance is to ensure that the auxiliary supply is always available.

In designing methods of cathodic protection, those who have specialised in corrosion engineering nowadays employ a whole battery of instruments for measuring site conditions before they decide on the type of protection to be employed. The acidity or alkalinity of the earth in the neighbourhood, under different conditions of moisture, has to be ascertained exactly; and specially developed voltmeters are used to measure with great precision the minute electromotive forces generated by the electro-chemical effects which cause corrosion. Changes in the degree of humidity present in the vicinity of both the metal to be protected and the anode from which the protective current is to flow, are of special importance. Also, the existence of "geological cells" must be ascertained. When a metallic structure running through the earth meets with differing conditions of acidity along its length, due perhaps to variations in the constitution of the soil, these differences may cause electrolytic cells to be set up, resulting in the passage of a current through the structure which may invalidate the cathodic protection measures.

The corrosion of the reinforcing used in concrete dams and other structures has been the subject of many papers before civil-engineering bodies in all parts of the world, and have emphasised the great care that is necessary in the choice of the cement used and in taking proper account of the chemical analysis of the water supply. In the future it may be that corrosion experts will play an even greater part in the design of this part of the project.

In all branches of engineering there is an ever-increasing urge to raise productivity, to eliminate waste, and to cut down the man-hours required for non-productive work. In pursuance of these aims the hydro-electric engineer may play his part by eliminating the waste of material—which, taken on a world-wide basis, is of gigantic proportions—due to the onset of corrosion. He may also make his industry still more of a paragon among power-generating organisations by reducing the already extremely small number of man-hours required for the production of electrical energy.

## St. Lawrence Seaway

IN a statement made recently Mr. Chevrier, Canadian Minister of Transport, made it clear that the Canadian Government was quite determined to proceed with an all-Canadian St. Lawrence seaway if the authorities in the United States found themselves unable to agree to a combined scheme. He emphasised, however, that Canada was fully committed to open the way for United States participation if this would not involve unconscionable delays in getting forward with the scheme. He thought the fate of the Wiley Bill most uncertain and unless this Bill was approved before Canada was in a position to start her own scheme the tortuous history of a co-operative effort would come to an end. In the meantime the U.S. Federal Power Commission has authorised the Power Authority of New York State to develop the American share of the two million horsepower that will become available at the International Rapids, but even this authorisation will require the sanction of the Court of Appeals in Washington before it can be legalised. The Wiley Bill was passed by the Senate of the United States some time ago and its opponents were expected to make every endeavour to get it rejected in the House of Representatives; it has now been approved, however, by the Public Works Committee of this House by a handsome majority and the next move would appear to rest with the Court of Appeals.

## Rural Electrification

THE Information Centre of the United Nations has issued two reports on rural electrification, in the preparation of which technical experts of some 18 nations have been engaged. Twenty-nine subjects have been singled out for special study, the following representing typical examples: Principles relating to the interconnection of smaller-sized hydro-electric plants to form local networks and linking these, in turn, to large power grids; thermal installations of small or medium power output for the combined production of power and process heat, using the latter for agricultural production and domestic needs; the operation of windmill generators in parallel with hydro or thermal plants; standardised voltages suitable for agricultural installations; extending the useful life of wooden distribution poles; the use of ground as a conductor; the use of electric light and ultra-violet ray to stimulate egg laying and the prevention of disease in stock; methods of financing rural installations, and so on.

The report is issued in two parts and copies are available from the Sales Section, European Office of the United Nations, Geneva, Switzerland; Part I, which contains 170 mimeographed pages, costs 6s. ($0.80), and Part II, 2s. ($0.30).

## B.C. Electric 1954 Programme

MR. A. E. GRAUER, President of British Columbia Electric Co. Ltd., has announced that their 1954 construction programme would be the largest for any single year in the company's history, the expenditure envisaged being $33·2 million. Heading the year's programme were two hydro-electric projects. The Bridge River plant (see WATER POWER, September 1953, page 343) would be completed in the summer by the commissioning of the fourth 62,000 h.p. generating set, bringing the plant's capacity to 248,000 h.p., and a start was to be made on the $8·5 million project at Seton Creek, due for completion in 1956.

## Record Power Output in Sweden

SWEDEN'S output of electric power totalled 22,500 million kWh in 1953, an all-time record. Out of the total, 9,500 million kWh, or about 45 per cent., are accounted for by the Power Board, while the remainder was produced by private or municipal companies. The most important addition during the past year was the Kilforsen plant in North Sweden, which has an annual output of 1,100 million kWh. The Power Board employs at present about 11,000 men on building projects, line work and for general operation. During the year the investments amounted to Kr.350,000,000 (£24,100,000) and the sales value of the power was Kr.260,000,000. Sweden's power re-

quirements are estimated to rise by about 1,200—1,500 million kWh per annum and the aggregate exploitable resources are calculated at 60,000 million kWh. About 65 per cent. of the total output of electric power is absorbed by industry, 25 by households, handicraft, small industry, etc., and the remaining 10 per cent. by the State Railways.

## Uganda Power Export

IT has been agreed in principle that Uganda will export bulk power from the Owen Falls power station to Kenya. Owen Falls is the power station that the Queen will open next April. Kenya has no source of power comparable to the Owen Falls and Uganda has a lack of large-scale users for the new station.

## Soviet Power Scheme

WORK is reported to be in progress on the large marsh area which stretches for more than 120 miles along the River Dnieper from Zaporoshye to Kakhovka, where an artificial lake, over 20 miles wide in some places, is to be formed as part of the great Kakhovka hydro-electric power scheme. The date set for the completion of this work has been brought forward from October 1 to April 1, 1954. Preparations have now been made to continue the work during the winter and new machinery has been brought in.

## New Canadian Power Source

A HYDRO-POWER reserve in Labrador-Ungava, twice the size of the Beauharnois, Quebec, development, may soon be in use. Prof. Hare, who is chairman of the geography department and director of the observatory, McGill University, said the hardest problem in Labrador-Ungava was the disposal of Hamilton River power in Newfoundland, but within easy transmission distance of the north shore of the St. Lawrence and the stream of ocean shipping. " The scale of this can be realised when compared with Beauharnois, which will have a capacity of 2 million h.p. and will be the world's third largest station. Grand Falls alone may ultimately produce twice this power and there are several other large sites on the river."

## Loan to Ceylon

IN London discussion has been concluded on the second stage of Ceylon's hydro-electric scheme between World Bank representatives and the Ceylon electrical department. Ceylon has requested the loan of 80 million rupees and Mr. Marshall, of the World Bank, will now submit his report to the New York authorities.

## Canadian Progress in 1953

THE Canadian Government is one of the few who publish annually a complete review of hydro-electric development in their country for the preceding year, and the report " Hydro-Electric Progress in Canada, 1953," now available free of charge from the Water Resources Division, Ottawa, as usual makes fascinating reading.

Although a tremendous amount of hydro-electric construction was under way in Canada in 1953, the amount of capacity brought into operation during the year was somewhat lower than that of recent years, although still appreciably above that of the pre-war period. New capacity in 1953 amounted to 638,012 h.p., bringing the total capacity of all water-power plants in Canada to 14,921,459 h.p., the net increase for the year being somewhat reduced by plant write-offs and list corrections. Plants and extensions under construction for operation in 1954 total 1,500,000 h.p. and those for later years about the same amount; also plans are being made to cover other large developments. Projects completed and under construction are widely distributed throughout the country, a number of them being located in rather remote districts.

Although water power continues to be the predominant source of electrical energy in Canada, an appreciable amount is also produced by thermal stations, and a number of thermal units of large capacity were brought into operation and others were under construction in 1953.

Total power consumption in 1953 was about 7 per cent. above 1952 but, generally speaking, all demands were being serviced although little reserve capacity was available. Some relief from the effect of peak loads was achieved by the completion of important interconnections between independent generating systems.

## Yukon Power Scheme

AN Eastern mining group is to build a pilot plant for a 1,000 million dollar power development in the Yukon. Ventures Ltd., and their subsidiaries, Frobisher Ltd., also Quebec Metallurgical Industries, have investigated the mineral and power potential of the Northern Territory. There are great possibilities for water storage in such lakes in the Upper Yukon River basins like Atlin Taku, Tagish Bennett, Teslin, and Labarge. Over a period of years 4,300,000 to 4,500,000 h.p. could be developed.

## Sicily-Italy Cable

THE cable across the Straits of Messina joining Sicily's power network to the Italian mainland will soon be completed. It will be 11,722 ft. long, the longest in the world, and will be swung without support across the Straits of Messina from two 728 ft. high steel towers. One is on the extreme tip of Calabria and the other on the Sicilian coast. Foundations have been laid using 7,000 tons of cement and 850 tons of steel. The cable will be slung at a minimum height of over 220 ft. above the sea and will not disturb shipping passing below.

## Kaplan Turbine Efficiency Tests

A PAPER presented to the Institution of Mechanical Engineers by Mr. S. P. Hutton, M.Eng., and intended for Written Discussion only, refers to the increasing difficulty and expense of making efficiency tests in the field and the growing tendency for purchasers to accept designs on the basis of model tests. Mr. Hutton considers that the use of the Ackeret formula in this connection cannot justifiably be applied to both Kaplan and Francis wheels with equal accuracy, and maintains that by restricting the argument to the Kaplan type, a more comprehensive analysis can be made, resulting in a formula of the Ackeret type but giving a more accurate measure of the true efficiency.

# Bolt-tension and Joint-reaction in Hydraulic Pipe-flanges

Gunnar Englesson, of Trollhättan, Sweden, analyses the conditions necessary for a pipe joint to be watertight under load and illustrates his thesis by means of a numerical example.

WHEN a flange is under internal pressure the bolt tension is balanced by the sum of hydrostatic end force and the joint reaction, as shown in Fig. 1. The determination of the bolting-up force that must be exerted on the packing to maintain a tight joint under service pressure conditions is not always easy. A graphical illustration of the bolt tension and joint reaction in a flange before and under internal pressure is shown in the diagrams Figs. 2 and 3. The triangle A, B, C, Fig. 2 represents the bolting-up conditions. The application of the bolting-up force $P_x$ causes an extension of the bolts amounting to say $\triangle l_1$ and a compression of the packing by an amount $\triangle t_1$.

In order to obtain the conditions when the internal pressure is applied, the hydrostatic force $P + P_1$ is set off on a line DE from the extension of AB, DE is drawn perpendicular to AC. A further increase of $\triangle l$ in the bolt extension takes place produced by the internal force $P + P_1$. This reduces the compression of the packing by an amount $\triangle t_2 = \triangle l$.

The compressive load on the packing will be further reduced by rotation of the flange. Let $\varphi_1$ denote the angle to which the flange bends under the action of the bolting-up force $P_x$ (see Fig. 3). When the internal force $P + P_1$ is applied the angle increases to $\varphi_2$ and the decrease of the packing compression will be equal to $\triangle t_3 = 2\,(\varphi_2 - \varphi_1)\,a_t$. The remaining compression of the packing will then be only $\triangle t = \triangle t_1 - \triangle t_2 - \triangle t_3$ corresponding to the force $P_t$ acting on the packing. As can be seen from Fig. 2, $\triangle t_3$ is set off from C in order to simplify the diagram. The values of $\triangle t_2$, $\triangle l$ and $\triangle t_1$ can easily be obtained from Fig. 2.

The determination of the angle difference $\varphi_2 - \varphi_1$ would be a rather complicated procedure by applying the exact theory of bending and deflection of circular plates. The simplified theory developed by Timoshenko, however, offers the designer a convenient method for practical applications, and I propose therefore to use it in relation to analysis of bolt tension and joint reaction in flanges.

According to Timoshenko the angle of a flange ring subjected to a bending moment $M$ per unit length of inner edge of the flange will be:

$$\varphi = \frac{12\, r_o\, k_1\, M}{E_f\, h^3\, \log_e \alpha} \qquad \ldots\ldots\ldots(1)$$

in which

$r_o$ = inner radius of pipe in cm.

$k_1$ is a coefficient dependent on the relation between

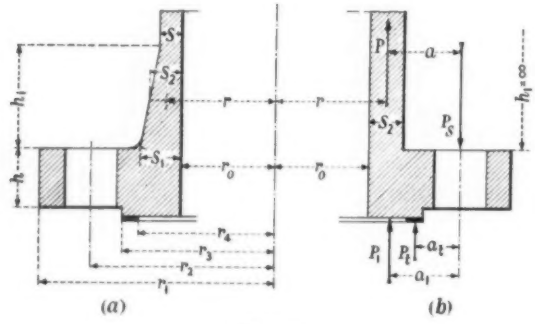



*Fig. 1*

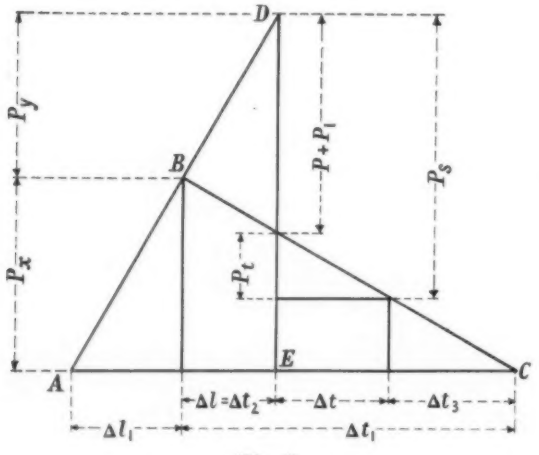



*Fig. 2*

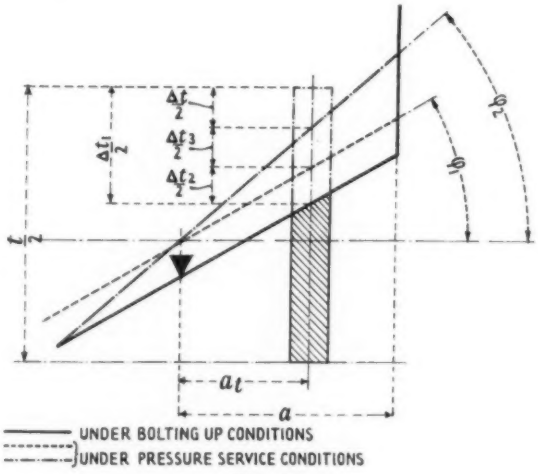



*Fig. 3*

the flexural rigidity of the pipe wall and the flange ring. If the rigidity of the pipe wall is increased in relation to that of the flange ring $k_1$ diminishes. Also in a series of flanges designed for the same internal pressure and equal strength, the magnitude of $k_1$ will decrease with the diameter of the pipe.

$E_f$ = Modulus of elasticity of flange in kg/cm²

$h$ = flange thickness in cm.

$\alpha = \frac{r_1}{r_o}$ where $r_1$ is the outer radius of the flange.

The coefficient of rigidity for a flange with hub according to Fig 1 will be found from the equation:

$$k_1 = 1 - \frac{1 + \frac{m\beta h}{2}}{1 + \frac{m\beta h}{2} + \frac{n}{2}\left(\frac{h}{s_2}\right)^3 \frac{\log_e \alpha}{\beta r_0}} \quad \ldots\ldots (2)$$

where $\beta = \frac{1{\cdot}285}{\sqrt{s_2 r}}$

and $s_2$ = mean thickness of hub wall with the radius $r$. The values of $m$ and $n$ are given in Fig. 4, from which

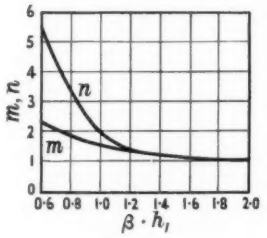



*Fig. 4*

it can be seen that $m$ and $n$ will be equal to unity when $\beta h_1 \geq 2$ ($h_1$ = height of hub).

With pretightened bolts the moment acting on the flange ring will be $P_x a_t$ and the angle becomes

$$\varphi_1 = \frac{12\, r_0\, k_1\, P_x\, a_t}{E_f\, h^3\, \log_e \alpha}$$

in which $P_x$ denotes the required bolt force to maintain a tight joint under service pressure conditions.

When the internal pressure is applied the angle will be

$$\varphi_2 = \frac{12\, r_0\, k_1\, (Pa + P_1 a_1 + P_t a_t)}{E_f \quad h^3 \quad \log_e \alpha}$$

$P$ denotes the tension in the pipe wall

$P_1$ denotes the tension between the flanges

$P_t$ denotes the compressive force on the packing under service

All forces in kg. per unit length of inner edge of flange.

$a$, $a_1$ and $a_t$ their respective moment arms in cm.

The shifting of the resulting bolt tension inwards caused by the inclination of the loaded flange is hereby neglected.

Then by using the relation $\triangle t_3 = 2\,(\varphi_2 - \varphi_1)\, a_t$ the decrease of the compression of the packing will thus be

$$\triangle t_3 = \frac{2 \times 12\, r_0\, k_1\, a_t\, (Pa + P_1 a_1 + P_t a_t - P_x a_t)}{E_f\, h^3\, \log_e \alpha}$$

Let $c\varphi = \frac{Moment}{Deflection}$ denote the rate of deflection of the flange rings produced by a unit moment then

$$c\varphi = \frac{E_f\, h^3\, \log_e \alpha}{24\, r_0\, k_1\, a_t} \quad \ldots\ldots (3)$$

Substituting $c\varphi$ in the former equation this will be represented in the following form:

$$\triangle t_3 = \frac{Pa + P_1 a_1 + P_t a_t - P_x a_t}{c\varphi} \quad \ldots\ldots (4)$$

The rate of extension $c_s = \frac{Force}{Extension}$ for the bolts produced by a unit force can be obtained from

$$\begin{cases} c_s = \dfrac{P}{\triangle l} \\ \triangle l = \dfrac{P\, 2\pi\, r_0\, l}{n\, A_s\, E_s} \end{cases}$$

Then $$c_s = \frac{n\, A_s\, E_s}{2\pi\, r_0\, l} \quad \ldots\ldots (5)$$

where $n$ = number of bolts

$l$ = length of bolt approximately $2\,h$ in cm.

$A_s$ = shaft area of bolt in cm.²

$E_s$ = Modulus of elasticity of bolt in kg./cm.²

In the same manner as outlined above the rate of compression $c_p = \frac{Force}{Compression}$ for the packing will be obtained from

$$\begin{cases} c_p = \dfrac{P}{\triangle t} \\ \triangle t = \dfrac{P\, 2\,\pi\, r_0\, t}{A_p\, E_1} \end{cases}$$

or $$c_p = \frac{A_p\, E_1}{2\,\pi\, r_0\, t} \quad \ldots\ldots (6)$$

in which $A_p$ = area of packing in cm.²

$E_1$ = modulus of elasticity of packing in kg./cm.²

$t$ = thickness of packing in cm.

The magnitude of the increase in bolt extension caused by the application of the internal load $P + P_1$ can be obtained from Fig. 2, from which

$$\begin{cases} c_s = \dfrac{P_y}{\triangle l} \\ c_p = \dfrac{P + P_1 - P_y}{\triangle l} \end{cases}$$

or $$\triangle l = \triangle t_2 = \frac{P + P_1}{c_s + c_p} \quad \ldots\ldots (7)$$

The compression of the jointing is shown in Fig. 5. As can be seen from the figure, it is assumed that the pressure is uniformly distributed over the whole area. This, of course, does not hold quite true, as the pressure will be slightly greater at the outer than at the inner periphery owing to the deflection of the flange.

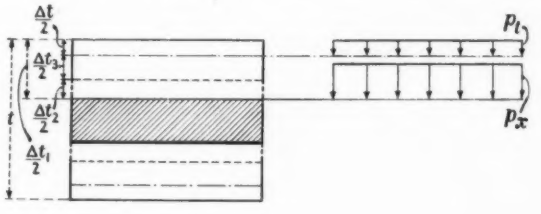



*Fig. 5*

The width of the packing, however, according to common practice, is small so that this assumption will not greatly influence the results.

The compression of the packing at bolting-up conditions will be

$$\triangle t_1 = \frac{P_x}{c_p} \qquad \text{.....(8)}$$

and the compression of the same at operating conditions will be

$$\triangle t = \frac{P_t}{c_p} \qquad \text{.....(9)}$$

Inserting the equations 4, 7, 8 and 9 in the relation

$$\triangle t_1 = \triangle t + \triangle t_2 + \triangle t_3$$

the following equation will be obtained

$$\frac{P_x}{c_p} = \frac{P_t}{c_p} + \frac{P + P_1}{c_s + c_p} + \frac{Pa + P_1a_1 + P_ta_t - P_xa}{c\varphi}$$

or

$$P_x = \frac{Pa + P_1a_1 + P_t\left(\frac{c\varphi}{c_p} + a_t\right) + \frac{P + P_1}{\frac{c_s}{c\varphi} + \frac{c_p}{c\varphi}}}{\frac{c\varphi}{c_p} + a_t} \qquad \text{.....(10)}$$

With a certain given compressive force $P_t$ on the packing under service conditions, the required bolt tension can be obtained from this equation.

The analysis is based on the assumption that the material of the packing follows Hooke's law. This holds exactly true only for rubber packings or packings containing much rubber as binding material. For flanges intended for use with water and oil Klingerit packings are commonly used. According to tests carried out by Babcock & Wilcox Ltd it appears that the deviation from Hooke's law is not very great, at least not for packings working at normal atmospheric temperature and having a thickness of $\frac{1}{32}$ and $\frac{1}{16}$ in.

It can be concluded from the test curves that the average value of the Modulus of Elasticity will be equal to $E_1 = 1500$ kg./cm.$^2$ using a thickness of $\frac{1}{16}$ in.

As the compression of the flange material, compared with that of a soft packing is very small, the former can be neglected without any substantial error. In case of a hard or very thin packing, it will be necessary to consider also the compression of the flange material. The combined rate of compression of packing and flange will then be

$$c_j = \frac{P}{\triangle t + \triangle f} = \frac{P}{\frac{P}{c_p} + \frac{P}{c_f}} = \frac{c_f}{\frac{c_f}{c_p} + 1} \qquad \text{.....(11)}$$

where $\triangle f$ is the compression of the flange material and $c_f$ the rate of compression.

Substituting $c_j$ for $c_p$ equation 10 becomes

$$P_x = \frac{Pa + P_1a_1 + P_t\left(\frac{c\varphi}{c_j} + a_t\right) + \frac{P + P_1}{\frac{c_s}{c\varphi} + \frac{c_j}{c\varphi}}}{\frac{c\varphi}{c_j} + a_t} \qquad \text{.....(12)}$$

The unit pressure on the packing produced by the bolt force $P_x$ will be:

$$p_x = \frac{P_x \, 2\pi \, r_0}{\pi(r_3^2 - r_4^2)} = \frac{2 \, P_x \, r_0}{r_3^2 - r_4^2} \qquad \text{.....(13)}$$

If the bolt force $P_x$ is known, the force exerted on the packing during operating conditions can be obtained from

$$P_t = \frac{P_x\left(\frac{c\varphi}{c_j} + a_t\right) - (Pa + P_1a_1) - \frac{P + P_1}{\frac{c_s}{c\varphi} + \frac{c_j}{c\varphi}}}{\frac{c\varphi}{c_j} + a_t} \qquad \text{.....(14)}$$

Assuming that the inclination of the loaded flange is neglected ($\triangle t_3 = 0$ in the relation $\triangle t_1 = \triangle t + \triangle t_2 + \triangle t_3$) the following equation will be obtained

$$\frac{P_x^1}{c_j} = \frac{P_t}{c_j} + \frac{P + P_1}{c_s + c_j}$$

or

$$P_x^1 = P_t + \frac{P + P_1}{\frac{c_s}{c_j} + 1} \qquad \text{.....(15)}$$

Dr. Ing. S. Berg has made up a chart based on this formula. (*Z.VDI* 1953 p. 349) For given ratios $\frac{P + P_1}{P_t}$ and $\frac{c_s}{c_j}$ the bolting-up force $P_x^1$ can be read off from this chart, covering all sizes of flanges. As the magnitude of the bolt force, however, depends not only on the ratio $\frac{c_s}{c_j}$ but also on many other factors the bolt force $P_x^1$ will be considerably smaller than by using the equation 12. The divergence will diminish with the rigidity of the flange. The bolt force $P_x$ according to equation 12 and $P_x^1$ according to equation 15 have been calculated for a series of flanges taken from the tables of the Swiss firm Clus. These forces in relation to the inner radius of the pipe are shown in Fig. 6. It can be concluded from the curve $\frac{P_x}{P_t}$ that the required bolt force will be comparatively much greater for a large than for a small flange, assuming the same unit compression of the packing at operating conditions. Further it can be seen that by neglecting the flange deflection the value of $P_x^1$ will be nearly the same for a large as for a small flange.

The decrease of joint compression $\triangle t_3$ caused by the flange deflection is for $r_0 = 100$ cm. about 10 times as large as for $r_0 = 7.5$ cm. The flanges are of the type shown in Fig. 1. The flange material is cast iron

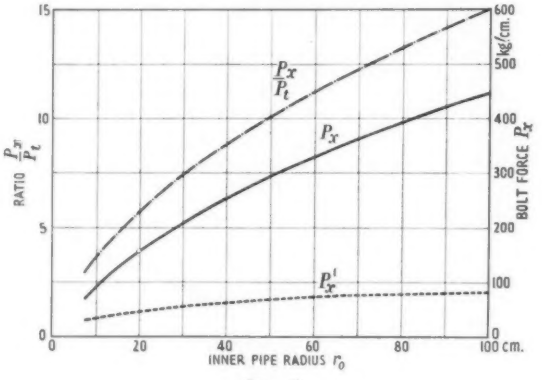



*Fig. 6*

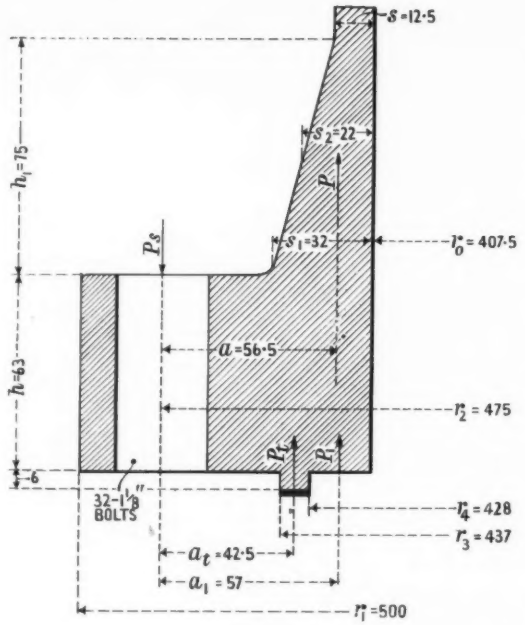



*Fig. 7*

and the internal pressure $p = 10$ kg./cm². The required pressure on the packing under service is assumed to be twice the internal pressure, therefore $p_t = 20$ kg./cm². Thickness of packing 0·16 cm. and $E_1 = 1500$ kg./cm².

A flange taken from "Modern Flange Design" Taylor Forge & Pipe Works U.S.A. has been chosen as a numerical example, see Fig. 7 and Table I. The compression of the flange material has been taken into consideration, and it is assumed that the area under compression of the flange will be the same as for the bolts. As $E_f = E_s$ then $c_f$ will be equal to $c_s$.

*Comments to Alternative 1.* The total required bolt force to maintain a tight joint will be equal to $430 \times 2 \times \pi \times 40.75 = 110000$ kg. and the total bolt force at operating conditions will be $(570 + 58 + 148) \times 2 \times \pi \times 40.75 = 200000$ kg.

The bolt tension produced by pulling up the bolts however, will depend in practice mostly on the judgment and the physical strength of the erector, and it is only by using torque wrenches that the designer can be certain of obtaining a desired bolt load. Excess loading is particularly to occur on flanges for low-pressure service, and due to overtightening of the bolts the flange and packing can be overstressed before the internal pressure is applied. The ASME Boiler Committee has therefore adopted the following formula to obtain the bolt force

$$P_x = \frac{A_{min} + A_{act}}{2 \times 2\pi \times r_0} \times \sigma$$

in which $A_{min}$ = total required cross section area of bolts at root of thread in cm². ($A_{min} = \dfrac{2\pi r_0 P_s}{\sigma}$

where $P_s = P + P_1 + P_t$

$A_{act}$ = actual cross-sectional area of the bolts at root of thread in cm².

$\sigma$ = maximum allowable stress of bolts in kg./cm².

The strength of the flange should thus be checked as follows:

(1) Before the internal pressure is applied and using the above mentioned formula for the bolt load.
(2) Under internal pressure.

For the numerical example $A_{min} = 143$ cm². and $A_{act} = 150$. Assuming $\sigma = 1400$ kg./cm². the bolt force will be:

$$P_x = \frac{(143 + 150) \times 1400}{2 \times 2\pi \times 40.75} = 800 \text{ kg./cm.}$$

and $p_x = 815$ kg./cm². The compressive force on the packing under service from the equation 14 will give $P_t = 520$ kg./cm. and $p_t = 530$ kg./cm².

According to information received from manufacturers of Klingerit jointings, this material remains elastic up to a compressive load of 1000 kg./cm.² at normal atmospheric temperature. The packing will therefore by no means be overloaded. The stress in the flange under service with $P_t = 520$ compared to that with $P_t = 148$, as assumed in the numerical example, will be as follows:

The external "normal" moment at operating conditions from $M = 570 \times 5.65 + 58 \times 5.7 + 148 \times 4.25 = 4180$ and with $P_t = 520$, $M = 5760$, thus an excess stress of about 38% in the flange. The total bolt load at service is equal to $(570 + 58 + 520) \times 2 \times \pi \times 40.75 = 294000$ kg. and the corresponding bolt stress 1950 kg./cm². The bolts are consequently considerably overstressed.

*Comments to Alternative 2.* With the computed value $P_x = 820$ the stress in the bolts will be just $\sigma = 1400$ kg./cm². The bending moment acting on

TABLE I NUMERICAL EXAMPLE

| *Symbol* | *Dimension* | *According to equation or figure* | *Numerical value* |
|---|---|---|---|
| $p$ | kg./cm². | | 28 |
| $p_t$ | kg./cm². | | 150 |
| $P$ | kg./cm. | | 570 |
| $P_1$ | ,, | | 58 |
| $P_t$ | ,, | | 148 |
| $P_s$ | ,, | | 776 |
| $\beta$ | $cm^{-1}$. | | 0.134 |
| $\beta h_1$ | | | 1.02 |
| $m$ | | Fig. 4 | 1.5 |
| $n$ | | ,, | 1.9 |
| $\log_e \alpha$ | | | 0.2 |
| $E_f = E_s$ | kg./cm². | | $2,15 \times 10^6$ |
| $A_p$ | cm². | | 245 |
| $A_s$ | ,, | | 6.4 |
| $l$ | cm. | | 13.0 |
| $k_1$ | | Equ. 2 | 0.32 |
| $c_\varphi$ | kg.cm./cm. | ,, 3 | 81000 |
| $c_s$ | kg./cm. | ,, 5 | 132000 |
| $c_f$ | ,, | | 132000 |
| *Alternative 1: Thick and soft packing* | | | |
| $t$ | cm. | | 0,16 |
| $E_1$ | kg./cm². | | 1500 |
| $c_p$ | kg./cm. | Equ. 6 | 9000 |
| $c_i$ | ,, | ,, 11 | 8400 |
| $P_x$ | ,, | ,, 12 | 430 |
| $P^1_x$ | ,, | ,, 15 | 186 |
| $P_x$ | kg./cm². | ,, 13 | 435 |
| *Alternative 2: Thin and hard packing* | | | |
| $t$ | cm. | | 0,04 |
| $E_1$ | kg./cm². | | 4000 |
| $c_p$ | kg./cm. | Equ. 6 | 96000 |
| $c_i$ | ,, | ,, 11 | 55700 |
| $P_x$ | ,, | ,, 12 | 820 |
| $P^1_x$ | ,, | ,, 15 | 338 |
| $p_x$ | kg./cm². | ,, 13 | 850 |

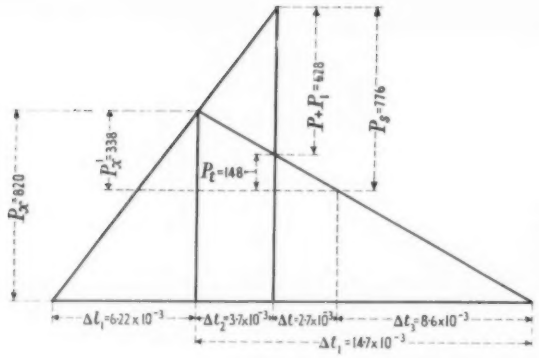



*Fig. 8*

the flange at bolting-up conditions will be $M = 820 \times 4.25 = 3500$ kg.cm. As the moment at operating conditions with $P_t = 148$ is $M = 4180$, the stress in the flange is correspondingly less, when the bolts are pretightened. Providing that the bolts are not pulled up with a torque wrench, it would be more advantageous to select a thin and rather hard than a thick and soft packing. Not only the flange but also the bolts will be less stressed, and there is reasonable protection against abuse due to overtightening.

For this alternative the compression of the packing and the extension of the bolts have been calculated as follows:

Acc. to equ. 8 $\triangle t_1 = \dfrac{820}{55700} = 14.7 \times 10^{-3}$ cm.

Acc. to equ. 9 $\triangle t = \dfrac{148}{55700} = 2.7 \times 10^{-3}$ cm.

„ „ „ 4 $\triangle t_3 = \dfrac{4180 - 820 \times 4.25}{81000} = 8.6 \times 10^{-3}$ cm.

„ „ „ 7 $\triangle t_2 = \dfrac{628}{132000 + 55700} = 3.4 \times 10^{-3}$ cm

Bolt extension $\triangle l_1 = \dfrac{P_x}{c_s} = \dfrac{820}{132000} = 6.22 \times 10^{-3}$cm.

These values as well as the values of the forces acting on the flange are shown graphically in Fig 8.

The method of calculating the bolt forces in flanges considered in this article shows that the required bolt force $P_x$ is considerably greater than the compressive load $P_t$ acting on the packing under service conditions. On flanges for low-pressure service ($p = 10$ kg./cm².) with an assumed pressure on the packing of $p_t = 20$ kg./cm². during operating conditions, the required pressure on the jointing can be as much as $p_x = 300$ kg./cm². for a pipe diameter of $2\,r_0 = 200$ cm.

When analysing flange stresses carefully, many of the basic values are already available, and it will therefore not be too cumbersome also to study the influence of the bolt force in relation to different jointing materials and their dimensions, thus avoiding the possibility of leakage trouble at site.

REFERENCES

(1) Taylor Forge & Pipe Works, U.S.A.: Modern Flange Design (1950 Third Edition).
(2) Berg: Die Schraube mit Vor- und Betriebslast *Z.VDI* Bd. 95 (1953) p. 349.
(3) Englesson: Analysis of Stress in Flanges for Turbine Pipelines. WATER POWER (1951) p. 335.

# Correspondence

### Calculating Flow by Relaxation Methods

*To the Editor of* WATER POWER

In the appendix to his paper on "Calculating Flow by Relaxation Methods" in the August 1953 copy of WATER POWER, Dr. Wright has made an erroneous statement. He says that the equilibrium equation for a function satisfying Laplace's Equation is

$$-4\psi_0 + \psi_1 + \psi_2 + \psi_3 + \psi_4 = 0$$

accepting an error of the order $h^2$. This should be an error of the order $h^4$.

If we expand the stream function as a Taylor series, we have (referring to Fig. 6)

$$\psi_1 = \psi_0 + \frac{d\psi}{dx} \cdot h + \frac{d^2\psi}{dx^2} \cdot \frac{h^2}{2} + \frac{d^3\psi}{dx^3} \cdot \frac{h^3}{6}$$

$$\psi_2 = \psi_0 + \frac{d\psi}{dy} \cdot h + \frac{d^2\psi}{dy^2} \cdot \frac{h^2}{2} + \frac{d^3\psi}{dy^3} \cdot \frac{h^3}{6}$$

$$\psi_3 = \psi_0 - \frac{d\psi}{dx} \cdot h + \frac{d^2\psi}{dx^2} \cdot \frac{h^2}{2} - \frac{d^3\psi}{dx^3} \cdot \frac{h^3}{6}$$

$$\psi_4 = \psi_0 - \frac{d\psi}{dy} \cdot h + \frac{d^2\psi}{dy^2} \cdot \frac{h^2}{2} - \frac{d^3\psi}{dy^3} \cdot \frac{h^3}{6}$$

On addition

$$\psi_1 + \psi_2 + \psi_3 + \psi_4 = 4\psi_0 + h^2 \left( \frac{d^2\psi}{dx^2} + \frac{d^2\psi}{dy^2} \right);$$

but, by Laplace's equation, this last term is zero, and so the relationship quoted above, viz.,

$$\psi_1 + \psi_2 + \psi_3 + \psi_4 - 4\psi_0 = 0$$

is an equilibrium condition including terms of the order of $h^3$.

If the governing equation is that due to Poisson

$$\nabla^2\psi = \text{Const.} = C$$

the equilibrium condition (to the order of $h^3$) becomes

$$\psi_1 + \psi_2 + \psi_3 + \psi_4 - 4\psi_0 - h^2C = 0$$

which again is easy to manipulate.

T. V. LAWSON
Lecturer in Aeronautical Engineering, University of Bristol

**British Standard for Protective Transformers.** The British Standards Institution has just published B.S.2046 'Protective transformers,' price 7s. 6d., dealing with the performance requirements and special characteristics applicable to current and voltage transformers intended for protective purposes in non-balanced protective equipment, and for the operation of earth fault devices with time-lag characteristics.

The standard is divided into three main sections; the first deals with matters which in general are common to both protective current and protective voltage transformers and includes clauses which define the various terms used and specify the nature of the high-voltage tests to be applied. The second and third sections deal more specifically with the requirements relating to current and voltage transformers respectively, e.g. with rated burdens.

*Fig. 1. General view of the Kemano camp looking north-west along Kemano valley*

# Kitimat

This project has the distinction of being the largest hydro-power development as yet built by private enterprise and it also comprises the largest underground powerhouse, the largest pressure conduits, the most powerful impulse turbines, and the greatest firm output of any single hydro station in the world.

PART ONE

COMMERCIAL production of a single pound of aluminium metal from raw bauxite ($Al_2O_3 2H_2O$) requires 10 kWh of energy, so that if this ubiquitous metal is to be produced at reasonable cost a cheap source of power must be used. Such is the modern industrial demand for aluminium that a greater volume of it is consumed than any other non-ferrous metal, but the virtue of its light weight causes it to fall short of copper, lead and zinc in actual tonnage. Aluminium production is thus a bulk process and requires very large blocks of power; at Kitimat a firm delivery of 1,670,000 h.p. will ultimately be available, with an installed capacity totalling 2,240,000 h.p., but the first stage of the development calls for a plant of 450,000 h.p. capacity. The project is being built by the Aluminum Company of Canada Limited, the largest subsidiary of Aluminium Limited, Montreal, Canada.

**Situation and Access**

Kitimat is situated about 500 miles north of Vancouver which forms a convenient base for operations, as much of the heavier material and equipment can be manufactured and shipped from there. Access to the various dams and other works at Nechako Canyon and Tahtsa Lake, together with some of the tunnelling

operations, is based on the Canadian National Railway line connecting Vanderhoof, Burns Lake and Prince Rupert, and a public road which runs from Burns Lake, Francois Lake and thence to Ootsa Lake. For the works at the outlet of Tahtsa Lake, the east portal of the power tunnels and, eventually, the Nanika-Kidprice stage of the development, the supply route consists of a road running from the west end of Francois Lake to the east end of Lake Tahtsa. On Tahtsa Lake itself transport is effected by 40-ton steel barges. To reach the head of the canyon at which the main Kenney storage and diversion dam is situated, a 60-mile road was made in the spring of 1951 and runs from Vanderhoof to the actual site. A landing strip suitable for medium-sized planes is also available at this site. A pier was completed at Kemano Bay in 1951 and a 10-mile road constructed to connect it with the powerhouse at Kemano.

The transmission lines will be accessible from tidal bases at Kemano Bay, Kitimat and the Kildala River, a road running from the latter place to the power station 30 miles away and also following the line to Kitimat 20 miles away. Over the Kildala Pass, this roadway could not approach all of the tower sites and the line was erected by means of cableways assisted by helicopter, of which considerable use has been made for surveys and emergency purposes. Access to the west end of the pressure tunnels and to the central adit to the inclined penstocks was effected by a heavy ropeway from the powerhouse site.

The Ice Age has left conspicuous marks practically everywhere one goes in Canada and the western

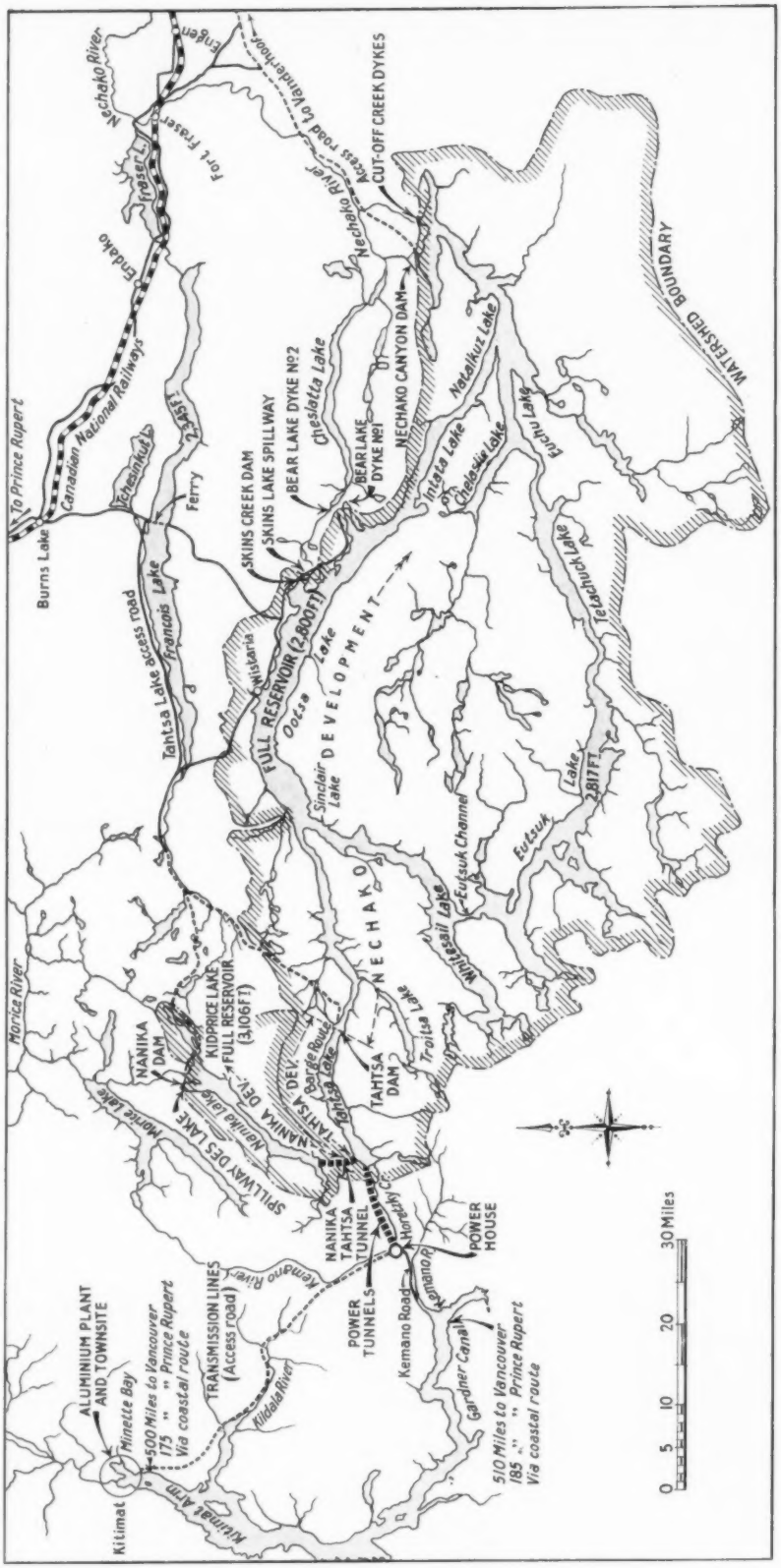



*Fig. 2. Sketch map showing Nechako-Kitimat hydro-electric scheme*

*Fig. 3. Helicopter landing at East Tahtsa camp*

slopes of the Coast Range have been heavily glaciated, the characteristically U-shaped valleys having been scoured down to well below sea level to form great fjords which carry deep water for many miles into the mountain mass. Ice has similarly been at work on the eastern slopes of the mountains and has given rise to a complex of long, narrow, deep lakes which in this case lie in favourable proximity to the fjorded coastal indent and are situated some thousands of feet above sea level. The lakes drain eastward and the fundamental conception of the Kitimat scheme lies in blocking up the eastern exits of the lake valleys and causing the entrapped water to flow down the steep western slopes of the mountains, in which relatively short pressure tunnels suffice to bring it down to sea level. Such are the circumstances at Kitimat that it has been possible, at the cost of erecting a few dams, not only to secure this diversion of flow but also to create a vast water storage, at a high head, which is capable of giving an extraordinarily high percentage of firm power.

A dam across the Nechako River has created a lake at an elevation of 2,800 ft. which will impound the run-off of the Eutsuk-Tetachuck-Euchu and the Tahtsa-Whitesail-Ootsa-Natalkuz drainage systems, the flow being directed through the Tahtsa Lake and River in reverse, and then carried by pressure tunnels to the underground power station situated within a few miles of an arm of the sea, viz., the Gardner Canal.

The higher westerly portions of the watershed receive an annual precipitation of 100 in. or more, mostly in the form of snow, while the lower easterly portions, from the head of the Ootsa Lake, receive about 20 in. The catchment area above the storage dam across the Nechako River is 5,475 sq. miles, and to this the 290 sq. miles of catchment connected with Nanika-Kidprice Lakes can be added. Flow data extending over a 20-year period (1930-50) indicate that the average discharge of the Nechako River system, neglecting the Nanika-Kidprice contribution, is 6,495 cusecs. The minimum monthly average flow during this period was 1,060 cusecs and the maximum 25,620 cusecs. On an annual basis, the minimum average over the 20-year period was 4,490 cusecs and the maximum 8,510 cusecs. Considered in relation to the component watersheds the average flow was:—

Tahtsa ... ... 940 cusecs
Nechako (excluding Tahtsa) ... ... 5,555 cusecs
Nanika-Kidprice ... 975 cusecs

To satisfy the first stage of the development it is estimated that 1,040 cusecs will be needed by May 1954 and this will exceed the firm supply obtainable from Tahtsa Lake to the tune of about 100 cusecs. Now that the Kenney dam diversion tunnel is in operation, however, an inflow will be provided that will not only suffice to make good this deficit but also fill the reservoir to spillway level by June 1957. It was for this reason that the Kenney dam diversion tunnel was begun early in 1951.

Neglecting the Nanika-Kidprice diversion, it is estimated that the level of the Nechako reservoir will be maintained at an elevation of 2,795 ft., with an overall variation of 6·5 ft. if average conditions are assumed. In minimum flow conditions, however, the reservoir may be drawn down about 17 ft. The elevation of the turbine nozzles in the power house is 210 ft. above sea level, so that the gross head—assuming an annual average reservoir level of 2,795 ft.—is 2,585 ft. At this head and an estimated regulated flow of 6,920 cusecs of water available, the power output will be 1,718,000 h.p. and it is proposed to erect 16 units giving a total installed capacity of 2,240,000 h.p.

The general scheme of impoundment will best be understood by reference to Fig. 2; the various storage, diversion and control dams result in the creation of 20 million acre-feet of reservoir capacity, the central feature of the scheme being the 317 ft. high dam across the Nechako canyon, which not only raises the original level of the lake series between 134 and 15 ft.

*Fig. 4. Rock working along Glacier Creek Road, showing the Atlas-Copco rock drill as used in the tunnels*

but also ensures the reversal of the flow through the entire system. This dam is situated about 16 miles downstream of the Natalkuz Lake, the site chosen being a point at which the river cuts through a ridge of volcanic flows and tuffs of Tertiary age. Here a relatively narrow gorge has been eroded out of the rock, which rises about 1,000 ft. above the general level of the plateau. At the axis of the dam, the upper surface of a series of relatively weak tuffs and other ash-derivative rocks lies 170 ft. below the river bed level, and above this series there is sequence consisting of 150 ft. of coarse volcanic breccia, 200 ft. of vessicular volcanic flows and finally a massive bulk of non-vessicular flows rises well above the crest

*Fig. 5. Aerial-ropeway skip ascending the mountain at Kemano*

**22½ × 25′ Diameter Tunnel**

*Typical Equipment and Performance Data*

Rock Drills: 21 "Copco" Jack Legs with Pneumatic Pusher.
"Jumbo": 19′ 6″, 3 Deck, on 60″ Gantry Rail.
Mucking Equipment: No. 100 Conway Loader.
Locomotives: 8-ton Goodman, Combination Battery and Trolley.
Cars: 6 yard, side dump, Granby Type. 36″ Gauge, 60″ Rail.
Drill Steel: ⅞″ Hexagon, "Coromant," Tungsten Carbide Bit Inserts.
Lengths of Holes Drilled: Cut holes 5′, 10′ and 15′. All others 13′.

| *Length of Steel Available* | *Bit Size in Inches* |
|---|---|
| 2·65′ | 1·34″ |
| 5·25′ | 1·30 |
| 7·90′ | 1·26 |
| 10·50′ | 1·22 |
| 13·15′ | 1·18 |
| 15·75′ | 1·14 |

Average feet drilled per grinding .. 60 feet
Average total life (footage) per bit .. 600 feet

Cycle:

| *Time in Hours and Minutes* | | *Typical Shift Heading Crew* | |
|---|---|---|---|
| Drilling | 1-27 | Shifters and Walkers | 2 |
| Load & Blast | 0-40 | Miners and Helpers | 22 |
| Ventilation | 0-24 | M.M. Operator | 1 |
| Mucking | 3-58 | Motormen | 4 |
| | | Electricians & Mechanics | 2 |
| Misc. | 1-27 | | |
| Total Cycle | 7-56 | Typical Shift Crew | 31 |

No. of holes drilled per round: 107 holes.
Total feet drilled per round: 1,345 feet.
Time period per foot of heading advance: 42 minutes.
Cubic yards per mucking hour: 57·2 × 2·2 tons = 125·8 tons.
Cubic yards per 11′ round: 220·0 c.y.
Cubic yards per man shift: 7·55 × 2·2 short tons = 16·6 tons.
Powder consumption per cubic yard 1″ × 8″, 40% Forcite Gelatine: 3·2 pounds.

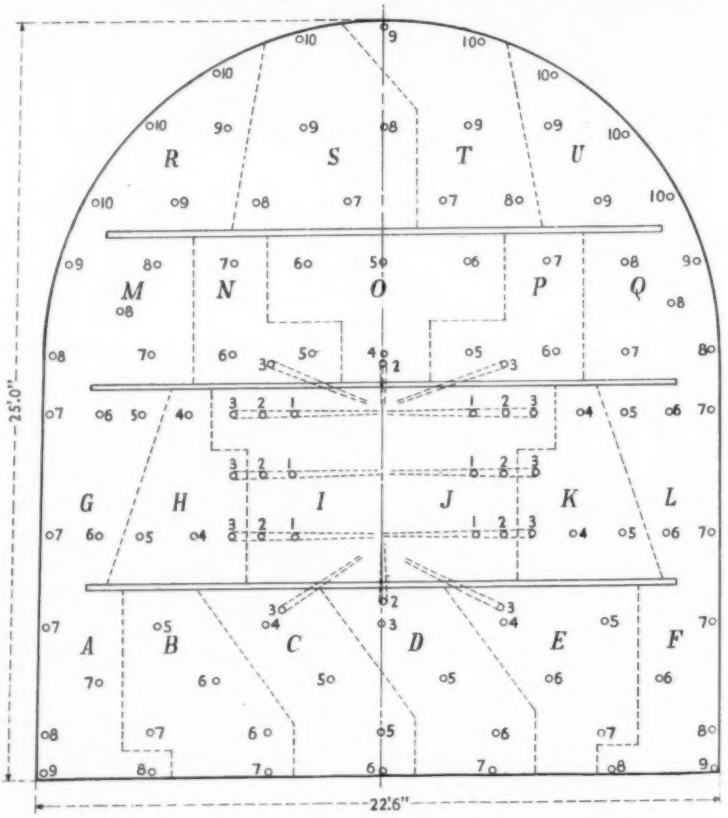



*Fig. 6. West Tahtsa tunnel, 22½ × 25 ft. section, drilling and blasting pattern and technical data*

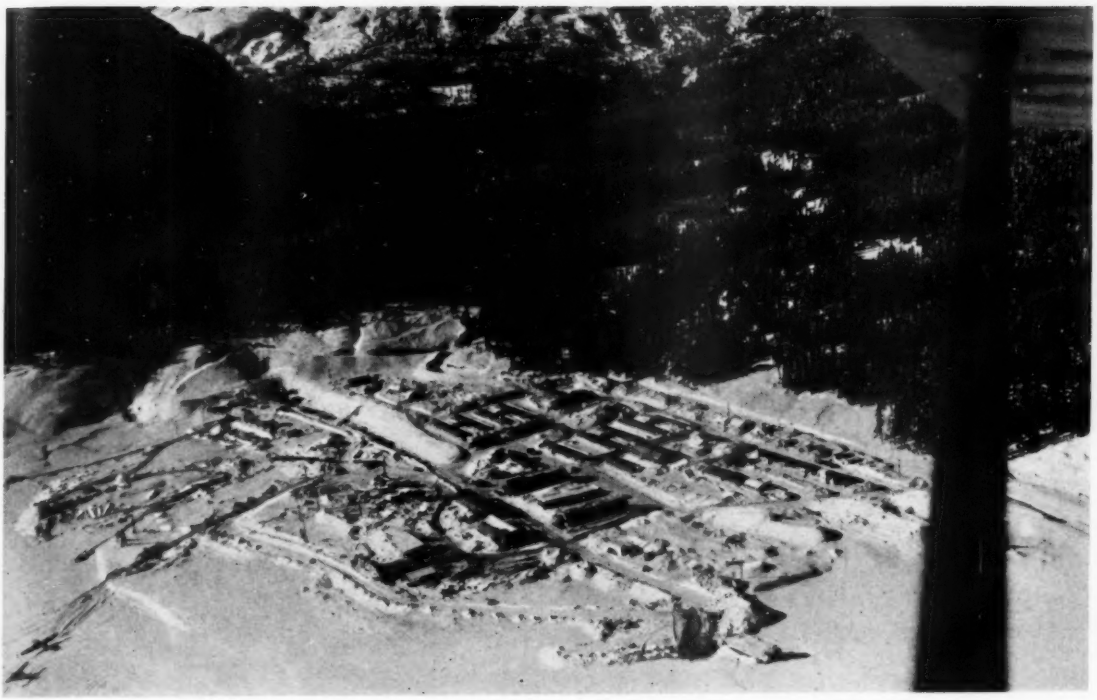


*Fig. 7. Aerial view of West Tahtsa Lake camp. Note open cut and construction adit portal*

level of the dam. A short distance above the site of the dam and running parallel with the volcanic ridge there is a felspar-porphyry dyke, of unfathomed depth, which acts as an admirable cut-off in the relatively weak tuffs.

The Kenney dam is of the rockfill type, the load-carrying element being a heavy fill, thoroughly sluiced, and supporting on the upstream slope an impervious section of rolled-earth construction enclosed between filter layers. Quarry-run rock is used for the upper section and provides the compacting load for the filter layers and impervious core, and also serves, of course, to protect the watertight section from wave erosion. The impervious core is carried up from the cut-off trench to nearly the top of the dam and has been thoroughly compacted by sheepsfoot rollers. The function of the downstream filter layers is to prevent the impervious core material from being forced into the rockfill interstices by the water pressure, the upstream filters fulfilling a similar function with respect to the upstream blanket of quarry-run rock. In the downstream section the filters comprise three layers; one of sand immediately downstream from the impervious core, a second of crushed rock or gravel 3 in. to $\frac{1}{2}$ in. in size, and third a layer of 10 in. to 3 in. material, this last adjoining the main rockfill. Apart from the layer of 10 in. to 3 in. material, the upstream filter is similar but, of course, in reverse. Under the impervious core, the rockhead has been grouted in depth to provide a secure cut-off. To do this, the first 25 ft. of depth was grouted at mild pressures (up to 20 lb. per sq. in.) and then to the final depth of 150 ft. by high-pressure holes drilled through the consolidated upper core.

This form of construction follows that adopted for the Nantahala dam which was constructed some ten years ago and is itself an improvement on a number of dams which have proved their reliability over a period of 60 years or more. A high degree of reliability is called for in the Kenney dam in virtue of its situation at the head of the Fraser River drainage system and the immense amount of water it impounds. The maximum height above bed rock is 317 ft. at full reservoir level of 2,800 plus 7 ft. flood discharge, being exceeded by San Gabriel No. 1 (380 ft.) and Salt Springs (328 ft.), both of which are in California. Freeboard at the abutments is 12·5 ft., with a super elevation of 5 ft. at the centre of the structure to provide for settlement. At crest level the dam is 1,550 ft. long and 40 ft. in width. Upstream the slope is 1 in 2·50, this inclination having been subsequently chosen to increase the overall factor of safety considerably beyond that which would have been obtained with the slope of 1 in 1·50 which was originally contemplated and which would have been adequate in ordinary circumstances. The slope of the downstream face averages 1 in 1·75, and is obtained by a series of repose angles and berms. Foundations were trimmed so as not to be steeper than 1 in 1·50 adjacent to the impervious core, in order to eliminate the possibility of differential settlement causing tension cracks to develop in the core. The longitudinal axis of the dam has been given a slight camber on the upstream face, this inducing a certain amount of axial compression and so eliminating any possibility of tensile stresses developing in the core.

Model tests made at the University of California on a dam with a crest width of 30 ft., an upstream slope of 1 in 1·50 and a downstream slope of 1 in 1·40 showed the construction to be entirely free from

*Fig. 8. Intersection between construction adit and No. 1 intake*

sliding movement even though the pressure on the upstream face was well over 10 times that which could arise from the impounded water; and tests on a shaking table model indicated that the dam would be perfectly safe with an acceleration equal to gravity—that is, ten times the usual allowance for dams in seismically active regions, although neither the Kenney dam nor the other structures associated with the Kitimat project are in such an area. The services of three leading consulting engineers were retained to consider the design—Dr. K. Terzaghi, Dr. J. P. Growdon and Mr. I. C. Steele—and it was concluded (1) that the foundations of the dam were of adequate strength and could be made watertight by grouting; (2) that the dam will have a high factor of safety; (3) that no maintenance would be required; and (4) that the life of the dam could be measured in geological time rather than historical time.

All the materials for construction were drawn from nearby quarries and borrow areas, both the rock and impervious fill being of excellent quality. All told about 3,700,000 cubic yards of material, weighing over six million tons, were used.

The site was unwatered by means of a diversion tunnel driven in the left bank. This was completed in 1951 and is of horseshoe section, 32 ft. in diameter and 1,430 ft. long. It was constructed with a slope of 0·0135 and was fitted at the outset with a closure sluice at the upper end, provision also being made for sealing it off permanently with a concrete plug in the grouted zone below the impervious core. To direct the water into the tunnel a rockfill cofferdam was erected above

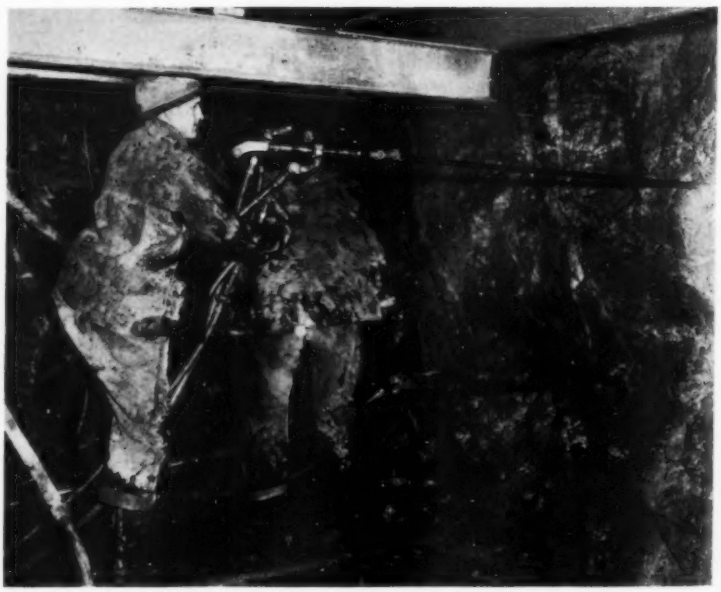


*Fig. 9. Atlas-Copco drills in action at the Tahtsa tunnel face on the first deck of the jumbo*

**25′ Diameter Tunnel**

*Typical Equipment and Performance Data*

Rock Drills: 16 "Copco" Jack Legs with Pneumatic Pusher.
"Jumbo": 22′ 0″, 3 Deck, on 60″ Gantry Rail.
Mucking Equipment: No. 100 Conway Loader.
Locomotives: 8-ton Goodman, Combination Battery and Trolley.
Cars: 6 yard, side dump, Granby Type on 36″ Gauge 60″ Rail.
Drill Steel: $\frac{7}{8}$″ Hexagon, "Coromant," Tungsten Carbide Bit Inserts.
Lengths of Holes drilled: Cut holes 5′, 10′ and 15′. All others 13′.

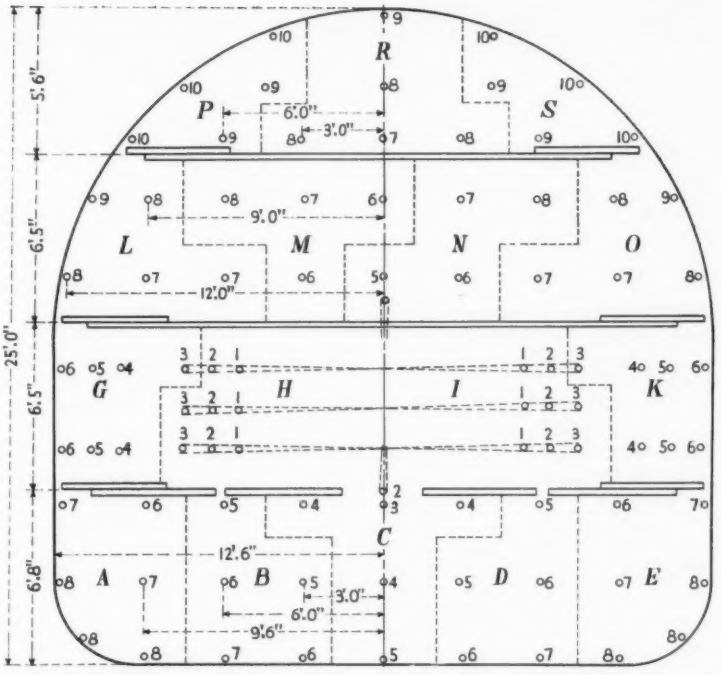
| *Length of Steel Available* | *Bit Size in Inches* |
|---|---|
| 5·25′ | 1·30″ |
| 10·50′ | 1·22 |
| 13·25′ | 1·18 |
| 15·75′ | 1·14 |

Average feet drilled per grinding .. 60 feet
Average total life (footage) per bit .. 600 feet

Cycle:

| *Time in Hours and Minutes* | | *Typical Shift Heading Crew* | |
|---|---|---|---|
| Drilling .. | 1-10 | Shifters and Walkers | 2 |
| Load & Blast | 0-55 | Miners and Helpers | 18 |
| Ventilation.. | 0-17 | M.M. Operator .. | 1 |
| Mucking .. | 4-10 | Motormen .. .. | 4 |
| | | Electricians & Mechanics .. .. | 2 |
| Misc. .... | 2-19 | | |
| Total Cycle.. | 8-51 | Typical Shift Crew .. | 27 |

No. of holes drilled per round: 92 holes.
Total feet drilled per round: 1,146 feet.
Time period per foot of heading advance: 48 minutes.




Cubic yards per 11′ 0″ round: 231·0 c.y.
Cubic yards per mucking hour: 52·2 × 2·2 = 120·0 tons.
Cubic yards per man shift: 7·60 × 2·2 short tons = 16·7 tons.
Powder consumption per cu. yd. 1″ × 8″, 40% Forcite Gelatine: 1·96 lb.

*Fig. 10. West Tahtsa tunnel, 25 ft. section, drilling and blasting pattern and technical data*

the site of the dam and had its crest at an elevation of 2,590 ft. Spillage capacity is available at Skins Lake where a spillway has been erected with an overflow section 70 ft. long at an elevation of 2,800 ft. and two 35 ft. by 35 ft. tainter gates. This structure is capable of discharging 58,000 cusecs with the

*Fig. 11. West Tahtsa No. 2 intake bore cemented to springline*

**25′ Diameter Horseshoe Tunnel**

*Typical Equipment and Performance Data*

Rock Drills: 15 Ingersoll Rand, DA 35's.

"Jumbo": 22′ 0″, 3 Deck, on 60 Gantry Rail.

Mucking Equipment: No. 100 Conway Loader.

Locomotives: 8 Ton Goodman, Combination Battery and Trolley.

Cars: 6 yard, side dump, Granby Type on 36″ Gauge 60 Rail.

Drill Steel: 1¼″ Round, "Atlas," carbon. Lug shank.

Bits: Detachable 2″ Carset, Tungsten Carbide.

Lengths of Holes drilled: Cut holes 13′ or 15′. All others 11′ or 13′.

| *Length of Steel* | *Bit Size in Inches* | |
|---|---|---|
| 5′ 8″ | 2″ to 1 13/16″ | New |
| 9′ 8″ | 1 13/16″ to 1 5/8″ | First gauge change |
| 11′ 8″ | 1 5/8″ to 1 3/8″ | Second gauge change |
| 13′ 8″ | 1 3/8″ to 1 1/8″ | Last grinding |
| 15′ 8″ | | |

(after grinding)

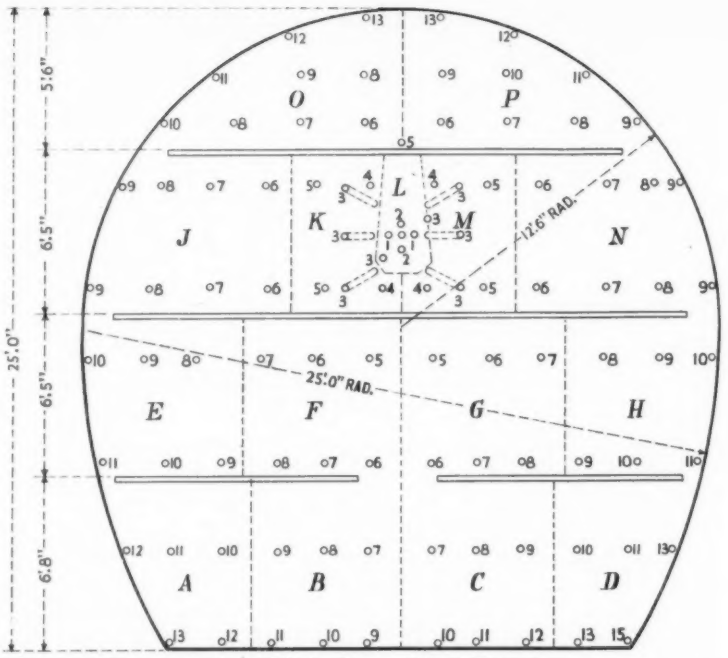
Gauge change after 60′ of drilling .. 60 feet
Average total life (footage) per bit .. 600 feet

Cycle:

| *Time in Hours and Minutes* | | *Typical Shift Heading Crew* | |
|---|---|---|---|
| Drilling | 1-35 | Walkers and Shifters | 2 |
| Load & Blast | 0-25 | Miners and Helpers | 31 |
| Ventilation | 0-19 | M.M. Operator | 1 |
| Mucking | 2-42 | Motormen | 2 |
| | | Electricians & Mechanics | 2 |
| Misc. | 0-09 | | |
| Total Cycle | 5-10 | Typical Shift Crew | 38 |

No. of holes drilled per round: 102 holes.

Total feet drilled per 10′ round: 1,148 feet.




Average advance per day (282′ in 6 days): 47 feet.

Time period per foot of heading advance: 31 minutes.

Cubic yards per 10′ 0″ round: 220 c.y.

Cubic yards per mucking hour: 87 × 2·2 = 190 tons.

Cubic yards per man shift: 9·68 × 2·2 short tons = 21·3 tons

Powder consumption per cubic yard 1¼″ × 8″, 60% Giant gelatine: 2·73 pounds.

*Fig. 12. Drilling and blasting pattern and technical data for 25 ft. horseshoe tunnel at Horetzky Creek; record week 282 ft. advance*

reservoir at flood level (2,807 ft.). The flood water will be discharged through the Cheslatta basin to the Nechako River about five miles below the Kenney dam.

Certain low points in the 2,800 ft. perimeter had to be made good by short earthfill dykes, for which, fortunately, suitable local material was available in every case.

The Kenney dam was closed off on October 8, 1952, and the reservoir will take 4½ years from that date to fill. In the meanwhile it will be possible to draw enough water from Tahtsa Lake, at the intake, to get the first phase of production into operation in the early summer of 1954.

In order to make the most effective use of the water in Tahtsa Lake during an exceptionally dry season in the first stage of development, a temporary rockfill embankment will be constructed across the outlet to raise the level of the water to 2,795 ft. Work on this embankment, which would be situated across the Tahtsa valley about 100 yards below the lake outlet would be scheduled to follow the completion of the power tunnel portals and intake structure. To enable water to be drawn from the Nechako reservoir to Tahtsa Lake it may be necessary to excavate a three-mile stretch of the river bed below the lake for a depth of 40 ft., but the actual depth of this cut will be governed by the power demand during the filling period of the main storage reservoir.

**Additional Storage Capacity**

Although there is no immediate intention of beginning any expansion before the main storage arrangements are completed, the impoundment of the Nanika-Kidprice lake system is also envisaged. This extension of the catchment area and storage capacity will necessitate a further large dam about half a mile below the outlet of the Kidprice Lake. Good foundation rock exists here in the form of andesite, but more grouting will be required for its consolidation than was necessary in the rock at Kenney. Some 500,000 cu. yards of material will be incorporated in this dam which will have a maximum height of 155 ft. and a crest length of 800 ft. Floodwater will be controlled by a low concrete free-crest spillway constructed at the westerly end of Des Lake, the discharge returning to the Nanika River several miles below the dam. The crest of the spillway will be at an elevation of 3,106 ft.

To carry the water from the Nanika-Kidprice system a free-discharge tunnel will be driven uphill from the Tahtsa portal. This will be 3·8 miles long and have an equivalent diameter of 13 ft.

**Main Power Tunnels**

Two pressure tunnels connect the intake at the west end of Tahtsa Lake to the underground power station at Kemano. Each has a length of slightly over 10 miles to the base of the surge chamber and where the

rock is sound no lining has been used apart from the concrete inverts. The intake is connected to Tahtsa Lake by a short channel, excavated in the dry, and 80 ft. wide at the invert. The intake structure is of concrete and is equipped with the usual trash racks, bulkhead guides and with two 14 ft. by 26 ft. fixed-wheel gates, each controlling the flow to one of the two pressure tunnels. The gate sill for the No. 1 tunnel will be at an elevation of 2,725 ft. in order to use the water from Tahtsa Lake in the first stage, but the No. 2 sill is at an elevation of 2,740 ft.

In analysing the economics of the power tunnels four possibilities were considered: the construction of a single tunnel 35 ft. in diameter; two 25 ft. diameter tunnels; a tunnel 30 ft. in diameter in conjunction with a second of 20 ft. in diameter; and two tunnels of 19 ft. and 30 ft. diameter respectively. On the grounds of both economy and reliability the driving of twin 25 ft. diameter tunnels was decided upon, spaced 300 ft. apart. The invert of the north tunnel has a slope of 0·0025 uniformly towards the surge shaft, the cross-sectional area being about 527 sq. ft. With overbreak the volume of rock excavated averages about 23 cu. yards per foot length, whereas theoretically 19·5 cu. yards only would be necessary. Each tunnel will carry 3,500 cusecs during normal operation but the designed maximum capacity is 4,500 cusecs. The tunnel was advanced from four working faces, one at each end and two more which were opened up from an access adit, 1,600 ft. long, driven from the Horetzky Creek valley at midpoint.

Even with four faces in operation the schedule was a very close one and the work at each heading had to be highly organised. The general procedure is similar at all four points, 100 h.p. Conway muckers being used for the loading and combination battery-trolley locomotives providing for the removal of the broken rock. Mining-type cars of 7 cu. yards capacity and operating on a 36 in. gauge 60 lb. track are provided, and the drilling work is done from fabricated steel jumbos fitted with bogey wheels for travelling on rails. The jumbos carry car-changing equipment, or "cherry picker," to feed the empties, in turn, ahead of the five-car train to the mucking machine which fills them in a matter of seconds. The work is organised on a six-day-week basis with three eight-hour shifts per day, but in the early days when the adits were being driven much of the work was carried out continuously through the whole of the week.

The Tahtsa camp had to be established on a somewhat inadequate alluvial bank projecting into the lake and the tunnel muck proved useful in developing an additional area of dry ground. The intake was designed to be about 60 ft. below the normal level of Tahtsa Lake and to eliminate the pumping and hoisting problems that would otherwise have been involved, an access tunnel was driven to intersect the main tunnel line 1,133 ft. from the portal. At this point of intersection the main tunnel was driven westwards towards Kemano and eastwards towards the intake, the break through to the intake end being protected by a cofferdam.

**25′ Diameter Horseshoe Tunnel**

*Typical Equipment and Performance Data*

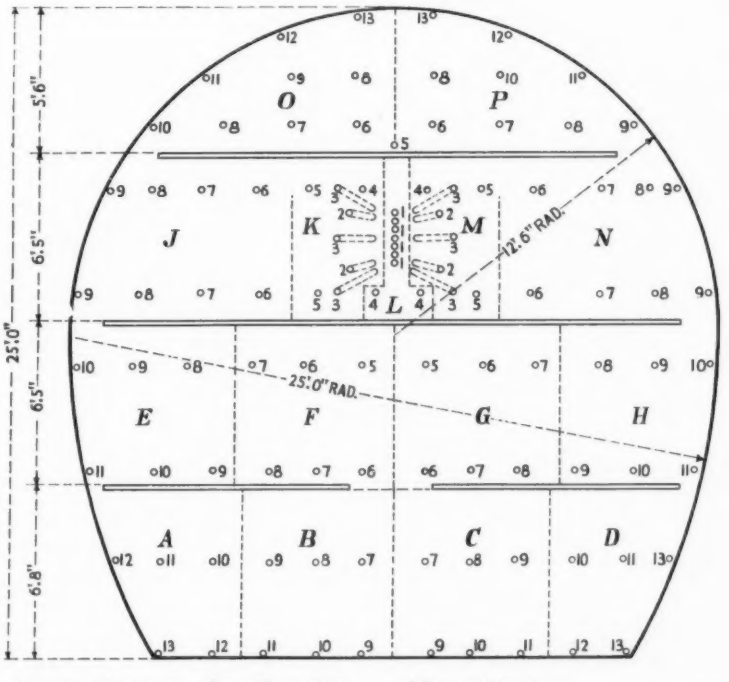
Rock Drills: 15 Ingersoll Rand, DA 35's.
"Jumbo": 22″ 0″, 3 Deck on 60 Gantry Rail.
Mucking Equipment: No. 100 Conway Loader
Locomotives: 8 Ton Goodman, Combination Battery and Trolley.
Cars: 6 yard, side dump, Granby Type on 36″ Gauge 60 Rail.
Drill Steel: 1¼″ Round, "Atlas," Carbon, Lug shank.
Bits: Detachable 2″ "Carset," Tungsten Carbide.
Lengths of Holes drilled: Cut holes 13′ or 15′. All others 11′ or 13′.

| *Length of Steel* | *Bit Size in Inches* | |
|---|---|---|
| 5′ 8″ 2″ | to 1⅛″ | New. |
| 9′ 8″ 1⅛″ | to 1″ | First gauge change |
| 11′ 8″ 1″ | to 1⅛″ | Second gauge change |
| 13′ 8′ 1⅛″ | to 1⅛″ | Last grinding |
| 15′ 8″ | | after grinding |

Gauge change after 60′ of drilling .. 30 feet
Average total life (footage) per bit .. 240 feet
Cycle:

| *Time in Hours and Minutes* | | *Typical Shift Heading Crew* | |
|---|---|---|---|
| Drilling .. | 1-40 | Walkers and Shifters | 2 |
| Load & Blast | 0-30 | Miners and Helpers .. | 33 |
| Ventilation .. | 0-23 | M.M. Operator .. | 1 |
| Mucking .. | 2-54 | Motormen | 2 |
| | | Electricians & Mechanics .. .. | 2 |
| Misc. .. | 0-48 | | |
| Total Cycle .. | 6-15 | Typical Shift Crew .. | 40 |

No. of holes drilled per round: 106 holes.
Total feet drilled per 9′ 8″ round: 1,184 feet.




Average advance per day (12 weeks): 37·63 feet.
Time period per foot of heading advance: 36 minutes.
Cubic yards per 8″ round: 211 c.y.
Cubic yards per mucking hour 73 × 2·2 = 160 tons.
Cubic yards per man shift 6·74 × 2·2 short tons = 14·8 tons.
Powder consumption per cubic yard 1¼″ × 8″, 40% Giant gelatine: 5·41 pounds.

*Fig. 13. Drilling and blasting pattern and technical data for 25 ft. horseshoe tunnel, Kemano to Horetzky*

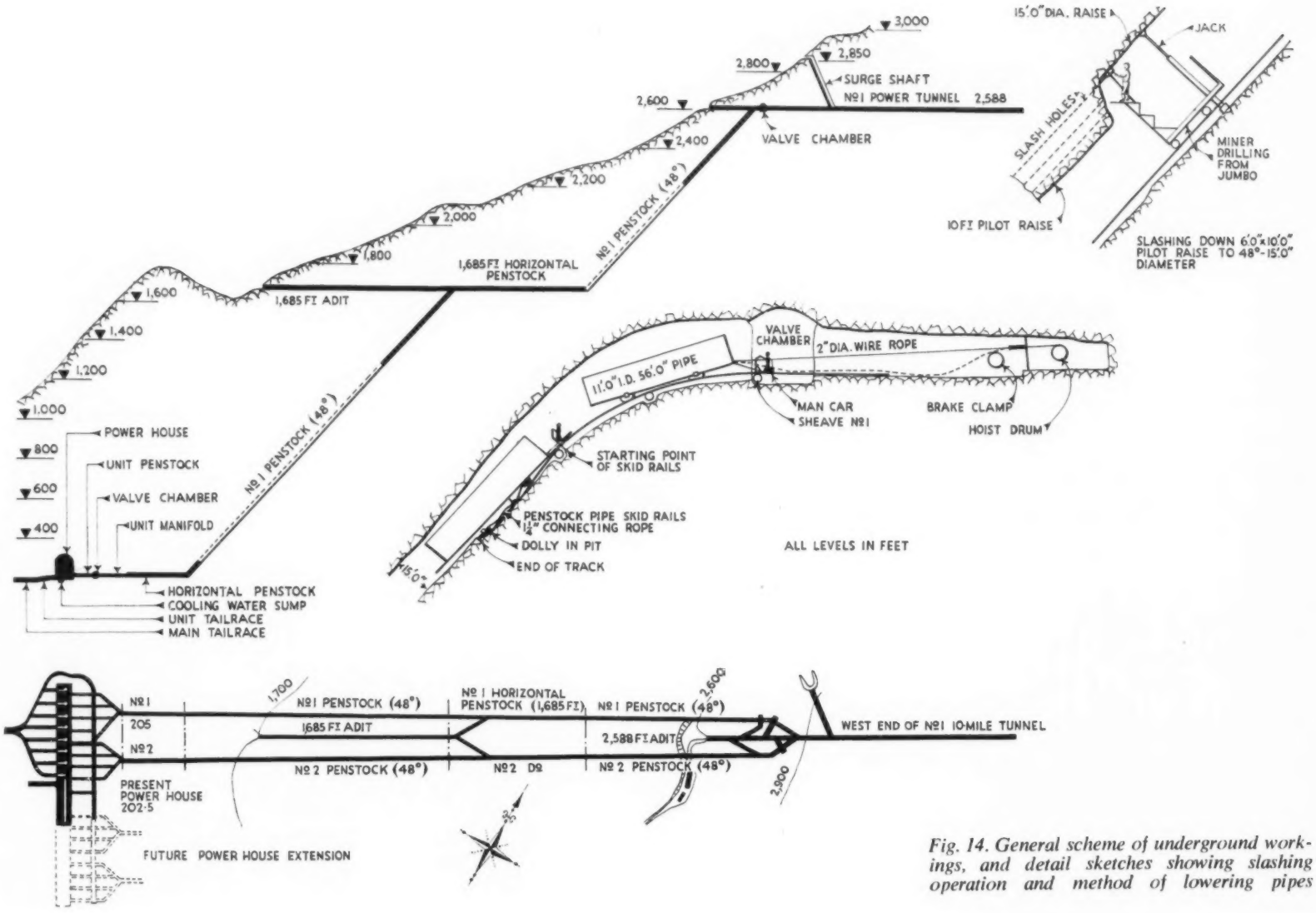



*Fig. 14. General scheme of underground workings, and detail sketches showing slashing operation and method of lowering pipes*

*Fig. 15. General view inside Horetzky adit*

For breaking the ground both the Swedish type of air leg drill and the heavy drifter drills normally used in North America were employed. Comparative studies made before the operations began revealed no conclusive evidence as to the superiority of one type or the other,* and the contractors—Morrison-Knudsen Company of Canada Limited—finally decided to use the lighter type of drill in the Tahtsa heading and the drifter drills in the other headings. For the light drills the steels were tipped with Coromant tungsten-carbide inserts and the heavy drifters with Carset tungsten-carbide tipped detachable bits. Experience has indicated that neither type has any clear advantage over the other but the rock conditions have admittedly been too variable for any valid comparison to be made. For example, on the Horetzky-Tahtsa section, 961 sets had to be erected, which was far in excess of the number of sets erected in any of the other sections.

The main intermediate adit at Horetzky was driven 1,700 ft. to the point of intersection with the tunnel line and encountered weak blocky rock. At the downstream end, at Kemano, on the other hand, the rock is a granodiorite and has been consistently strong. The access tunnel here was driven into the mountain for a distance of 450 ft., parallel to and between the two penstock tunnels, and forked out at the inner end to meet the main pressure tunnels. Few fault zones have been encountered, but where they do occur steel support arches have been erected.

All the main tunnels are being driven full face and the three westerly headings are being drilled with Ingersoll Rand DA35 drifters. The jumbos have three operating decks and are provided with vertical columns, arms and standard-type dump and swing equipment. Drilling patterns are adapted to the character of the rock, the accompanying line sketches (Figs. 12 and 13) showing typical patterns and the delay firing sequence. For the 25 ft. wide horseshoe section from 102 to 108 holes are normally driven to a depth of from 11 to 13 ft., resulting in an advance of from 10 to 12 ft. per round. Where the ground is weak, however, the hole depth is shortened and steel supports are erected as close to the face as possible.

The steel used with the DA35 drifters is $1\frac{1}{4}$ in. round Atlas, cut into 5 ft. 8 in., 9 ft. 8 in., 11 ft. 8 in., 13 ft. 8 in., and 15 ft. 8 in. lengths and finished with lug shanks. The Carset tungsten carbide bits are 2 in. in diameter and a new bit is invariably used on the 5 ft. 8 in. starter steel. Gauges have been found to reduce by about $\frac{1}{16}$ in. per change and the bits are discarded when they have been reduced to $1\frac{11}{16}$ in. Normally they are reground after 60 ft. of drilling but in the hardest rock encountered they have had to be reground after 30 ft. of penetration. Bit life has been found to range between 240 and 600 ft., representing eight to ten grindings. Compressed air is provided at 100 lb. per sq. in. by 4,000 cu. ft. per min. capacity diesel or electrically driven compressors in each heading.

For the Atlas-Copco RH656 drills used in the Tahtsa heading a Coromant chisel bit is provided with a tungsten carbide insert permanently set in the $\frac{7}{8}$ in. drill rod. Holes are started with a 1·34 in. diameter 2·65 ft. long steel and are subsequently carried through changes of 4·25 ft., 7·90 ft., 10·50 ft., 13·50 ft., and 15·75 ft., the bit diameter being reduced to 1·14 in. diameter for the longest steel. Average life ranges from 400 to 600 ft., depending on the hardness of the ground, and corresponding to from eight to ten regrinds.

The normal working crew for each of the three west headings consists of two walkers and shifters, 31 miners and helpers, one mucking machine operator, two motormen, one electrician and one mechanic, making 38 in all. At the Tahtsa heading the number per crew depends on whether the jumbo is manned with 21 or 16 machines; with 21 machines, 22 miners and helpers are employed and the crew totals 31 persons, but if 16 machines only are used the number is reduced to 27. Only one driller is required for the "Copco" drill, whereas each drifter requires a driller and a helper. Blasting is effected by electric delay-type detonators used in conjunction with $1\frac{1}{4}$ in. by 8 in. 40 per cent. Giant Gelatine dynamite. Time delay intervals are shown in Table I.*

TABLE I

| No. of delay | Explosion time from ignition | Length of cap | No. of delay | Explosion time from ignition | Length of cap |
|---|---|---|---|---|---|
| | sec. | in. | | sec. | in. |
| Instantaneous | 0·0 | 2 | No. 8 | 4·65 | 2 |
| No. 1 | 0·8 | 2 | No. 9 | 5·35 | 2 |
| No. 2 | 1·25 | 2 | No. 10 | 6·10 | 3·25 |
| No. 3 | 1·70 | 2 | No. 11 | 7·00 | 3·25 |
| No. 4 | 2·20 | 2 | No. 12 | 8·00 | 3·25 |
| No. 5 | 2·75 | 2 | No. 13 | 9·15 | 3·25 |
| No. 6 | 3·35 | 2 | No. 14 | 10·50 | 3·25 |
| No. 7 | 4·00 | 2 | No. 15 | 12·30 | 4·75 |

* "Tunnel and Powerhouse Excavation at Kemano." Paper presented to the Canadian Institute of Mining & Metallurgy by Franklin T. Matthias and C. W. Abrahamson, respectively Assistant Manager and Assistant Resident Engineer of the B.C. project of the Aluminium Company of Canada, at a Meeting held in Vancouver on April 15, 1953.

**Ibid.*

*(Continued on page 112)*

# Iowa Fifth Hydraulics Conference

We present a series of specially contributed abstracts of the proceedings of the Fifth Hydraulics Conference held at the Iowa Institute of Hydraulic Research, which was devoted to the general subject of sediment transportation.

PART TWO

**Relation of Suspended-Sediment Concentration to Channel Scour and Fill, by Luna B. Leopold and Thomas Maddock, Jr.**

During the passage of a flood the channel of an alluvial stream scours and fills with considerable rapidity. Though such occurrences may be random, it seems more likely that there is a definite pattern of channel change, directly related both to discharge and to the sediment load supplied to the river by its drainage basin. An analysis was made of concurrent values of suspended sediment load, width, mean depth, mean velocity, and discharge at a number of gauging stations. Each of the first four was then expressed in terms of a simple power function of $Q$ for analysis and comparison. Studies were made of the San Juan River at Bluff, Utah; Colorado River at Grand Canyon, Arizona; and Rio Grande at Bernalillo, New Mexico, the values of the coefficients and exponents in the power functions being evaluated.

Although each of the streams investigated had its own particular curves and coefficients, the scour and fill of each bed during flood appeared to be an adjustment of channel shape in response to varying sediment load. The adjustment took place rapidly, but showed some time lag. An example of the comparison between sediment load and discharge is given by the Colorado River at Grand Canyon. For a discharge of 20,000 cusecs on the rising stage, the sediment load was 1,000,000 tons per day; for the same discharge on the falling stage the sediment load was only 120,000 tons per day. The smaller load was accompanied by a definite increase in depth and decrease in velocity, whereas the width remained the same.

The authors concur with other investigators that the channel roughness, as expressed in the Manning coefficient $n$, decreases with increased concentrations of suspended sediment.

**Observations on the Nature of Scour, by Emmett M. Laursen**

From a definition of scour and the principle of conservation of matter, a primary characteristic of all scour phenomena is demonstrated—that the rate of scour is equal to the difference between the capacity for transport out of the scoured area and the rate of supply of material to that area. Assumptions describing certain aspects of the flow are sufficient to establish further characteristics which should apply to most cases of local scour—that the rate of scour will decrease as the flow section enlarges, that there will be a limit to the extent of scour and that this limit will be approached asymptotically.

The effectiveness of these principles is demonstrated for the case of scour by a submerged horizontal jet (Fig. 7). The first characteristic can be stated in the form of a differential equation, the integration of which requires expressions for the local transport capacity and the rate of supply. For an approximate empirical capacity function (and a zero rate of supply) the differential equation is integrated to give a relationship between the extent of scour and time. Experimental data confirm the analysis.

The usefulness of the general concepts in interpreting experimental results is illustrated for the case of scour around bridge piers and abutments. The experimental observations that velocity of flow and sediment size have no measurable effect on the equilibrium depth of scour are rationalised from the premise that the transport capacities in the unobstructed flume and in the scour hole have the same relationship to the mean velocity of flow. The equilibrium then does not depend on the absolute rate of transport but only on the balance between the two capacities. The same reasoning is used in explaining why the depth of flow has an influence on the equilibrium depth of scour. The simplification of the problem that has been obtained indicates that correlation between field and laboratory should be possible, since the absolute rate of transport does not need to be scaled.

**Deposition at the Heads of Reservoirs, by Alfred S. Harrison**

Experience has shown that there are two characteristic types of sediment deposits in reservoirs along alluvial rivers: (1) those occurring generally over the reservoir bottom, which are mostly composed of the finer silts and clays; and (2) those occurring as deltas at the head of the reservoir, which are mostly sands and gravels. Delta deposits progress in two directions from the upstream end of the reservoir—downstream into the reservoir, and upstream, progressively aggrading the river above the limit of reservoir backwater, as shown in Fig. 8. Deposit within the reservoir lessens its capacity, whereas upstream deposition increases backwater effects.

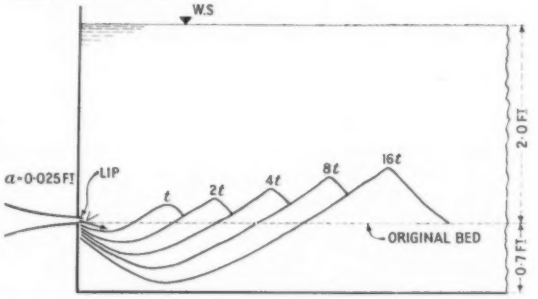



*Fig. 7. Diagram of experimental equipment for scour by submerged jet*

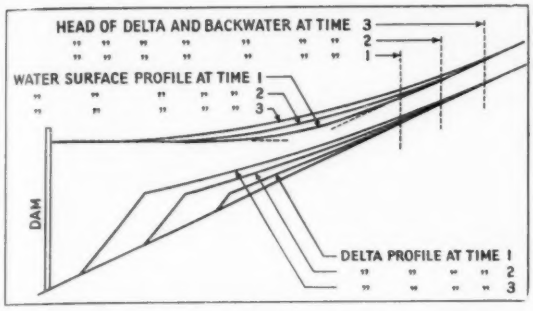



*Fig. 8. Progressive growth of idealised reservoir delta*

The sediment load in an alluvial stream is divided into two classifications, bed-material load and wash load. The former is derived primarily from the bed; the latter, which is finer than the bed material, is derived from land-surface wash. Bed-material load begins to deposit immediately on entering the reservoir, the larger grains depositing first. The wash load deposits more generally throughout the reservoir, frequently moving as density currents down to the face of the dam. It has been observed that the main thread of stream flow tends to remain concentrated for a considerable distance into the reservoir, building a delta along its course, although it may shift to different locations from time to time. Silts and clays, when deposited and consolidated, are highly resistant to erosion. Delta deposits generally are stratified vertically into layers of sand and silt.

An empirical approach to the computation of delta profiles is presented which is based on the changing transport capacity of the stream. It is recommended that information be obtained at various reservoirs on water-surface profiles, bed samples, bed-material distribution, cross sections of the delta surface, surveys of channel alignment, and reconnaissance after major floods.

**The Transportation and Deposition of Sediments in Estuaries, by C. F. Wicker**

An estuary is defined as the tidal portion of a river in which the direction of flow reverses and the water-surface elevation rises and falls with a periodicity corresponding to that of the ocean tide. If sedimentation in estuaries is uncontrolled, ship channels shoal, sheltered anchorages disappear, and the estuary ultimately becomes a marsh. The economic importance of these phenomena is consequently very great.

Estuaries are shoaled primarily by the detritus carried into them from the rivers entering them, though littoral drift may play a significant role. Sediment is transported into and through the estuary by currents, waves, and density effects, acting in concert in an extremely complex fashion. The combination of upland run-off and tidal cycles is significant in estuarine regimen. A salinity wedge may create excessive turbulence at its interface and materially affect sediment transport. Flocculation also occurs in the saline water.

From consideration of sediment movement through an idealised or "laboratory" estuary, the essential features of the complex motion of the sediment can be presented. There are many complications not encompassed in such a simplified estuary, principally the fact that upland run-off to the estuary is highly variable, as are the tides, from season to season.

Accurate quantitative observations of the sediment in transport at various reaches in an estuary are uncommon. Less is known of such movement than of sediment transport in rivers. Study of a shoaling problem must be based primarily on a general knowledge of the hydraulics of the waterway, characteristics of the material in transport, and field observations of transport. Although the conclusion that most estuaries are doomed to destruction seems inescapable, accurate and detailed knowledge of the occurrences will aid in extending their usefulness.

**Effect of Shape on the Fall Velocity of Gravel Particles, by M. L. Albertson**

Considerable evidence has been found that the terminal fall velocity is the most important characteristic of sediment particles. A characteristic which affects the fall velocity is the shape of the particle. Because of the infinity of variability for such an abstraction, an important part of the problem is the definition and measurement of shape. A shape factor found to be significant is $c / \sqrt{ab}$ in which $a$, $b$, and $c$ are, respectively, the longest, intermediate, and shortest dimensions of the particle.

Using the equipment shown schematically in Fig. 9, measurements of fall velocity for a number of individual particles were made for gravel obtained from the river, from a rock crusher, and from a glacial moraine.

For a large number of natural particles comparisons were made between the projected area and the product $ab$ and between the volume and $abc$. A sufficiently good correlation was found for the former to make possible its use; the latter, on the contrary, was not wholly adequate.

The coefficient of drag was found to be a function of the Reynolds number and the shape factor $c / \sqrt{ab}$.

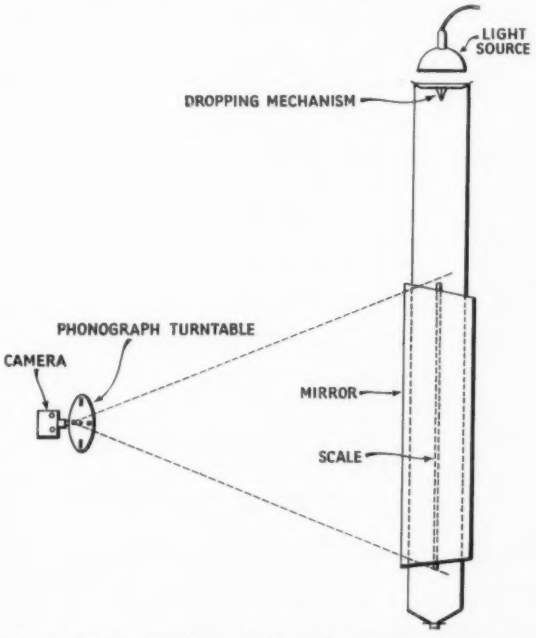



*Fig. 9. Schematic diagram of equipment for measuring fall velocity*

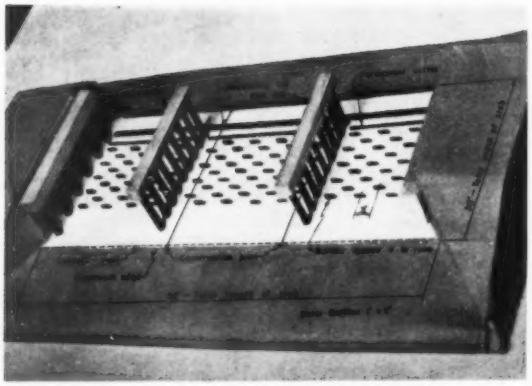


*Fig. 10. Model of turbulence flume, Middle Loup River at Dunning, Nebraska*

*Fig. 11. Collecting sample with U.S. DH-48 sampler at measuring sill on Middle Loup River*

In calculations of Reynolds number and coefficient of drag, use of the nominal diameter in place of $\sqrt{ab}$ is recommended. Although considerable scatter in the results was found, the fall velocity of a particle is markedly dependent upon its shape.

**The Measurement of Total Sediment Load in Alluvial Streams, by P. C. Benedict and D. Q. Matejka**

Measurement of the total sediment load in transport by a stream has long been a perplexing problem. Suitable methods for the measurement of suspended load have been perfected, but comparable techniques for bed load have not yet been developed. At present total loads are based on measurements of the suspended portions and estimates by means of empirical formulas of the bed load. A new technique, which is the topic of this paper, is the measurement of suspended load at sections in which all of the material is placed in suspension either naturally or artificially at constricted sections for which the velocity and degree of turbulence are high.

For two of the natural sites selected for study the turbulence at the contracted section was sufficient at all stages to transport the bed load in suspension. At two others the turbulence generated was not always sufficient, but provided valuable comparisons nonetheless. For the Middle Loup River at Dunning, Nebraska, at which site an artificial turbulence flume was constructed (Figs. 10 and 11), the applicability of several bed-load formulas was investigated by study of records obtained. Differences from 30 to 78 per cent. were found between measured and calculated bed-load transports.

It is recognised that the total sediment load will remain constant from section to section in a graded alluvial stream. If the velocity or turbulence changes in a reach, the proportion of bed to suspended load will change also. Considerable study will be required before a dependable bed-load formula is developed which will give consistent results in natural channels which are in equilibrium, not to mention the more difficult conditions for streams which are aggrading or degrading.

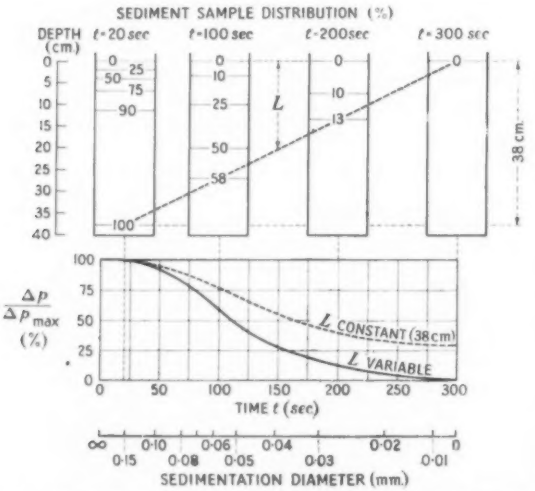



*Fig. 12. Differential pressure record during sedimentation of typical river sediment in column having traversing piezometer*

**An Instrument for Rapid Size-Frequency Analysis of Sediment, by David W. Appel.**

A new instrument for size-frequency analysis, based on a stratified-suspension technique, has been developed. The sample of sediment is initially supported on a filter of sintered brass at the top of the sedimentation column. As the filter is withdrawn, the sediment falls freely, forming a stratified suspension. At the same time a continuous measurement of differential pressure is made, from which the fractional weight of sediment and its fall velocity can be obtained directly.

A schematic record of the differential pressure between two fixed piezometers 38 cm. apart during sedimentation of a typical river sediment is shown by the dotted line in Fig. 12. However, in order to shorten the time for the complete analysis, the lower piezometer is made to traverse upward through the suspension at a uniform rate so that the length of the column decreases to zero in five minutes. The resulting pressure record is shown by the solid line in the figure.

A newly-developed very sensitive differential-pressure cell is used with a pen recorder to obtain an automatic recording of the size frequency of the sediment, thus reducing to as little as eight minutes the time required for a single operator to analyse a sediment sample. The accuracy of the instrument compares favourably with other available instruments.

# Progress in New Zealand

Excerpts dealing with hydro-electric matters are made from the Annual Report of the Commissioner of Works recently presented to New Zealand Parliament.

BY way of introducing this report, the Hon. W. S. Goosman, Minister of Works, stated that development of hydro-electric power in both the North and South Islands continued to be accorded high priority by the Government. In the North Island power was generated at the Maraetai station in October 1952 by operating under partial head. At the Whakamaru project, also on the Waikato, the river had been diverted and substantial progress made, although the work was somewhat retarded by late delivery from overseas of certain plant. So far as the proposed Atiamuri scheme was concerned, good progress had been made on the design, which was in the hands of Sir Alexander Gibb and Partners, of London. In regard, however, to the proposed scheme at Ohakuri on the Waikato River, the Ministry of Works was now in a better position to undertake this work, and it had been decided that the design would be completed by the Department, which also had the design of the Waipapa scheme under way. The demand for power continued to grow, and accordingly the Government was undertaking investigations in other watersheds to ascertain what further sources of hydro-electric power were available for economic development in the North Island.

So far as the South Island was concerned a contract for the completion of the major scheme on the Clutha River at Roxburgh was let on July 24, 1952, to the lowest tenderers. Holland & Hannen and Cubitts Ltd., in association with the Swiss firm of S. A. Conrad Zschokke. The work was actually handed over to the contractors on September 29, 1952. The firm had brought to New Zealand 82 English and Swiss staff and 322 workmen. They had been faced with certain difficulties in establishing themselves and had had to move more slip material than was originally contemplated. Nevertheless, present indications were that better progress would be necessary if the target date for completion was to be realised.

Progress on the extensions to the Waitaki power station had been good, and if certain overseas equipment came to hand as expected additional power should be available from this station next winter. At Lake Tekapo work necessary to impound a further 15 ft. depth of water, which could be drawn upon when required, was well in hand and should be finished in time to meet next winter's demand for power. Work at the Cobb project was progressing satisfactorily, and here again additional power should be available for next winter.

**Report of the Engineer-in-Chief**

The fuller statement on hydro-electric development is published in the Report of the Engineer-in-Chief, Mr. C. W. O. Turner, M.E., B.Sc., M.I.C.E., who states that the year 1952-53 was one of vigorous progress. Three major new projects (Maraetai, Whakamaru, and Roxburgh, two major extension works (Cobb and Waitaki), three lake-control works (Pukaki, Tekapo, and Waikaremoana), and thirteen substations were under active construction.

Investigations were also being conducted on nine prospective future projects, six on the Waikato River (at Waipapa, Atiamuri, Ohakuri, Parariki, Aratiatia, and Huka), one at Braeburn in the Murchison district, another at Benmore on the Waitaki River, and also lake control works at Hawea. The development of the Kaituna River (fed by the Rotorua lakes system), the erection of further substations at Wellsford, Thames, New Plymouth, and Opunake, and the development of power from natural steam at Wairakei were also under examination.

Investigations of sites suitable for hydro-electric projects were made at Waipapa, Atiamuri, Ohakuri, Parariki, Aratiatia, and Huka, on the Waikato River in the North Island, and at Braeburn and Benmore in the South Island, also for lake control at Lake Hawea. New areas for hydro-electric power were reconnoitred and further data compiled on the hydrological and power resources of the country as part of a long-term development programme. The next area proposed

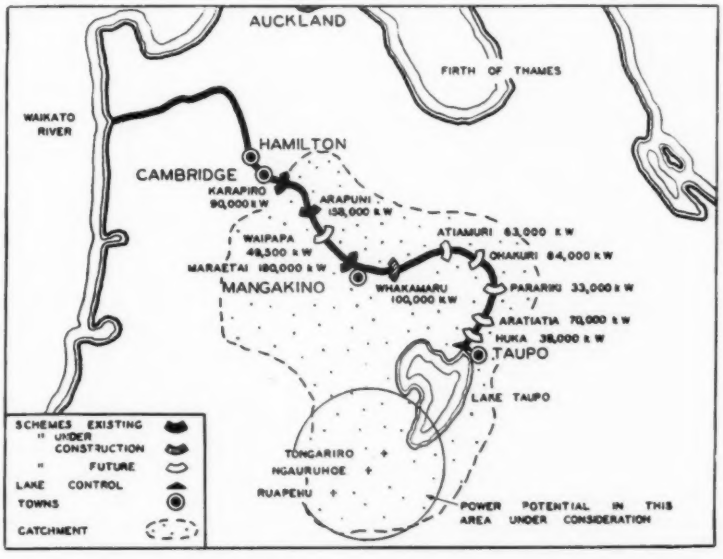



*Fig. 1. Waikato River development, total potential 1,216 MW*

for detailed investigation is the Kaituna River system.

The design resources of the Department were engaged mainly on the production of plans for the Whakamaru, Cobb, Waitaki, and Roxburgh projects and for the Islington substation. Design work was also commenced on the Waipapa project (downstream from Maraetai on the Waikato River), on the Braeburn project, and on new substations at Wellsford, Thames, New Plymouth, and Opunake.

Construction work during the year centred on the three big projects in hand at Maraetai, Whakamaru, and Roxburgh. Maraetai is in operation under partial head, Whakamaru is about half completed, and construction is well under way at Roxburgh. Major extensions to the power stations at Cobb and Waitaki were continued. Lake control works were completed at Pukaki and continued at Tekapo and Waikaremoana.

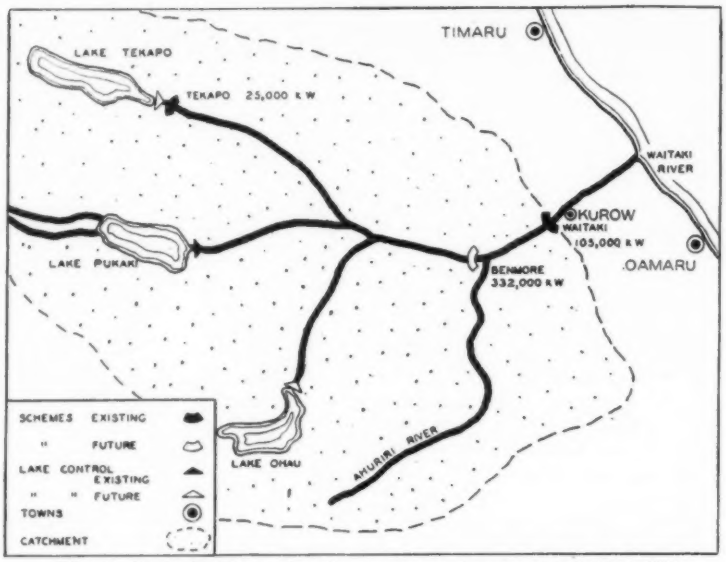



*Fig. 2. Waitaki River development, total potential 1,600 MW*

**Waikato River Basin**

*Maraetai:* Three of the five units have been installed at this station, which commenced feeding into the North Island system under partial head on October 31, 1952.

The large concrete plug at the junction of the sloping leg of spillway tunnel and of the tunnel initially used for diversion of the river was completed ahead of schedule. The spillway intake was also completed and the gates and lifting equipment installed. However, poor workmanship in the intake-gate lifting cylinders delayed the final completion of the station. Examination of the faulty work was facilitated by the use of industrial X-ray and gamma-ray equipment..

Notable phases in the construction schedule at Maraetai were as follow:—

Construction operations began in a small way at Maraetai in December 1945. There was much initial work in establishment of road access and in building of Mangakino construction township.

| | |
|---|---|
| Diversion tunnel completed ..... | March 1949 |
| River diverted ..... | May 14, 1949 |
| Concreting in river ..... | Begun June 1950 |
| Arch dam completed ..... | September 1952 |
| Powerhouse ..... | Begun December 1950, completed August 1952 |

Quantities of material involved were:

| | |
|---|---|
| Concrete in arch dam (cu. yards) ..... | 148,150 |
| Concrete in powerhouse (cu. yards) ... | 57,622 |
| Total concrete (cu. yards) ... | 205,772 |
| Drilling for grout (ft.) ..... | 476,545 |
| Cement injected into country (tons) ... | 12,760 |

*Whakamaru.*—The design work for Whakamaru is nearly completed. Contracts were let during the year for the preparation of the steel plates of the main penstocks, and for the supply of intake screens and for the elevator. The river was diverted in November 1952 and the foundation area dewatered.

Excavation and grouting of foundation rock and construction of sections of the concrete dam on both banks of river are proceeding; 15,700 cu. yards of concrete have been placed so far. Excavation to foundations for the earth wing-dams on both sides of the river was started; 136,500 cu. yards of rock, gravels, sand, etc., were excavated this year.

Progress was seriously delayed by the late arrival of two radial cableways required for shifting materials on the site.

*Waipapa.*—A report prepared on the sites examined in the Waikato River at this point indicates that a hydro-electric project of some 50,000 kW could be constructed here. Additional drilling is being carried out on the selected site for the purposes of providing information on foundations to assist the designing engineers.

*Atiamuri.*—The design of this project was let to the British consulting firm of Sir Alexander Gibb and Partners, whose proposals were examined from time to time during the year. Site investigation continues in order to obtain necessary data.

Information on certain special methods of grouting the alluvial formations was supplied to the consulting engineers.

*Ohakuri.*—Preliminary investigation is almost completed. Some grouting tests in the rock abutments are being carried out to supplement design data.

*Pararíki.*—Field investigations and drilling revealed foundation conditions unfavourable for economic development of any of the three available sites.

*Aratiatia.*—Topographical surveys were completed and drilling for foundation investigation carried out, also incidental access roading and bridging provided.

*Huka.*—Mapping of this area was completed, and drills are operating on both banks to obtain geological and engineering data concerning the foundation rock.

**Kaituna River Development**

Because of engineering difficulties at Pararikí, a study of the Kaituna River was made and a possible power development scheme considered. A probable power site is being investigated by drilling.

**Waikaremoana Lake Control**

The sealing of leaks in the bed of the lake was continued and the leakage flow reduced. The low lake-level presented an opportunity for excavating rock outcrops in the intake channel, and consolidating the leak-sealing blanket "in the dry."

The dry winter together with the low lake-level also facilitated construction of the siphon inlet and barrel, now 50 per cent. completed.

The highway culvert-control structure and stilling basin are now also under construction.

**Braeburn Development (Marlborough), No. 1 Project**

The preliminary investigation of this proposed project was completed and is being studied for design purposes. The proposal envisages leading the water of Lake Rotoroa by a six-mile tunnel to the Tutaki Valley, for power generation by discharge into the Tutaki River. Drilling is in hand to determine the best sites. An automatic recording river-gauge has been set up on the Mangles River to obtain additional hydrological data.

**Cobb Project**

The design of the earth dam at Cobb was completed. Standards for controlling the quality of materials used during construction are being defined. The detailed designing of the second penstock proceeded throughout the year.

The spillway-block structure, containing 16,800 cu. yards of concrete, was completed in March 1953. Good progress was made with the two spillway culverts and the stilling basin. The spillway system is now almost completed.

The stockpiling of material for the earth dam and of river gravel for the drains and dam shoulders was continued. A total of 195,000 cu. yards of material was stockpiled.

Excavation for the dam foundation commenced in August 1952 and the placement of the dam material began in January 1953. The amount placed to date is 9,300 cu. yards out of a total of 440,000 cu. yards. The dam when completed will have a maximum height of 114 ft., a crest length of 713 ft., and a crest width of 20 ft.

The erection of the second penstock, which is about half finished, is being carried out over very steep and difficult terrain. The welded sections are now subjected to X-ray examination.

Powerhouse erection was held up by late steel deliveries, but the new powerhouse is now 85 per cent. completed and totally encloses the old structure. The control block and workshop buildings were completed structurally; finishing work is in progress.

Preparations for the installation of the first machine at the Cobb project are in hand.

**Waitaki River Basin**

*Lake Tekapo Control Works.*—The main power station has now been successfully operating for two years. All design work for the control works was completed during the year.

Work on the dam site was suspended over the period January-April 1952 to enable the lake level to be lowered sufficiently to guarantee continuity of construction on the site. To date 100,000 cu. yards of material have been excavated from the dam site and river channel and stockpiled for the nearby highway deviation. All concreting below bed level was completed, and the control structure is now beginning to take shape. Approximately 75 per cent. of the total concrete has to date been placed.

*Lake Pukaki Control Works.*—Work during the year was confined mainly to road deviations and some remedial and maintenance works in and around the control structure. The camp has now closed down. The buildings from it will be transferred to Lake Hawea.

*Waitaki Power Station Additions.*—Complete construction drawings were supplied to the contractors (Messrs. Downer-Morrison Knudsen Co.) early in the year. The contractors started work in May 1952. Progress on the additions has been good, and there is every indication that they will become power producing by the winter of 1954.

*Benmore Power Project.*—Field work was resumed and grouting tests continued to determine the permeability of the site and the most suitable method of grouting. The data obtained are now being studied. Additional hydrological investigations were made into the catchment area of the Waitaki River to determine the increased power potential of this project, having regard to the control works now in operation at Lake Pukaki and the future storage that will be available at Lake Tekapo.

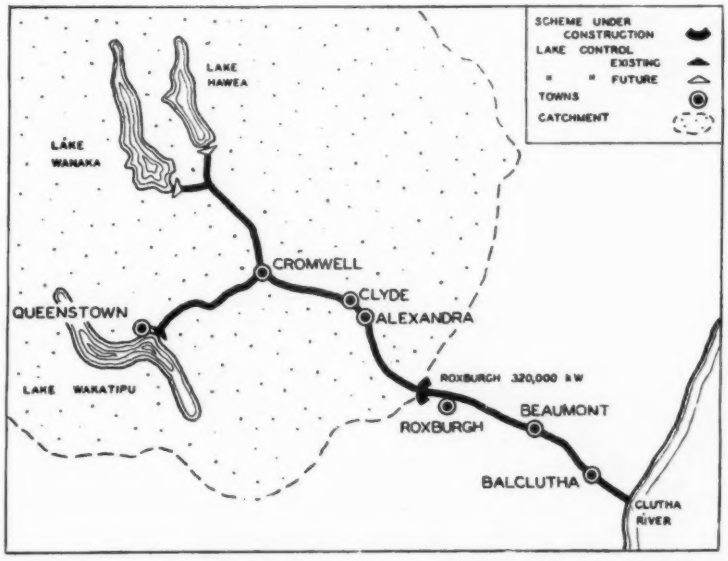



*Fig. 3. Clutha River development, total potential 1,140 MW*

**Clutha River Basin**

*Roxburgh Power Project.*—On September 29, 1952, the completion of the Roxburgh project was handed over by the Ministry of Works to a combine of the British firm of Holland & Hannen and Cubitts Ltd., and the Swiss firm of S.A. Conrad Zschokke. The combine is known as Cubitts and Zschokke. The contractors took over the whole job as a going concern, including all labour, plant and stores on the site. The Department's staff continued in a supervisory capacity until the taking over of plant and stores was completed, when departmental staff were reduced to those required in connection with the general administration of the contract. Some officers were made available to the contractors for a period.

During the year the work at Roxburgh was concentrated on the diversion channel and the installation of the main construction plant. A total of 149,800 cu. yards of material was excavated and 39,500 cu. yards of concrete were placed. The construction of temporary village areas was practically completed. Grouting operations in the spillway area were almost completed; blanket grouting of the sluice-block area was started; while grouting in the river bed at the site of the two main cofferdams, with the intention of sealing off the foundations, is nearing completion.

The secondary concreting plant has been in operation since April 1952. The main concrete plant (four 2-cu. yard mixers) was completed and has been in operation since March 1953. The main aggregate processing plant is in course of erection. At the railway station a cement tippler with two 500-ton cement storage silos has been constructed. The two cableways (1,605 ft. span) were completed in October 1952.

At the request of the contractors and to expedite the work, the Department's staff helped with the erection of the main plant and with grouting.

The detailed design for the Roxburgh project kept well ahead of the contractor's construction requirements and also enabled tenders to be called for subcontracts such as the fabrication of the penstocks, the intake gates, and the powerhouse superstructure.

The total number of workmen employed on the Roxburgh project was: March, 1952, 535; September 1952, 770; March 1953, 1,073.

*Hawea Lake Control.*—Preliminary designs were studied up to the limit of the site information available.

**Future Hydro-Electric Development**

Records of the flow from lakes and rivers suitable for future development, together with those already under development, have been compiled and statistical data prepared. Liaison was maintained with the State Hydro-Electric Department and the Soil Conservation and Rivers Control Council, and programmes were prepared for future recording stations on rivers suitable for development. Automatic records from the Mangles and Rangitaiki Rivers became available during the year. A hydrological investigation was made into the storage possibilities of Lake Taupo, as part of the Huka Falls and Aratiatia projects.

An investigation was made into the potential of the Waikato River below Karapiro with a view to the possible development of the remainder of the fall of this river to the sea. The upper Waikato River was also investigated with the object of determining the possibilities of developing further the power available in this area and also augmenting the present intake of water from Lake Taupo. It appears that a substantial block of power is likely to be available from the Upper Waikato River, and extensive field work to determine the economics of this appears to be warranted.

Reconnaissance surveys were carried out to determine the optimum use of the power potential of the Upper Rangitaiki River, the Rotorua lakes, and the Kaituna River area. A comprehensive plan of this area was prepared for further study. A similar plan for future investigation is under preparation in respect of the Southland lakes, including Lakes Te Anau, Manapouri, Monowai, Hauroko, and Poteriteri.

A reconnaissance of possible power projects was also made during the year in North Canterbury, Marlborough, and Buller.

## World's Longest Under-Water Power Cable

British Insulated Callender's Cables Limited announce receipt of an order valued over $3,000,000 from the British Columbia Electric Company for the supply and installation of some 77 miles of 138 kV gas-pressure-type cable. This cable will be laid across the Georgia Strait for transmission of power from the mainland to Vancouver Island to provide an adequate supply for future development. The cable route, to be determined more exactly by marine surveys this spring, will be somewhere between Greater Vancouver and a point south of Nanaimo on the Island of Vancouver where it will join the British Columbia Electric System.

This outstanding power scheme, due to be in operation in 1956, will transmit electricity under water at a higher voltage and at a higher carrying capacity than any other similar installation in the world.

The anticipated route includes continuous lengths of cables each approximately 17 miles laid under water at depths up to 600 ft. Four single-core 138 kV cables will be required—three for regular use and one as a reserve—the circuit being able to transmit approximately 115 MW. The cable will be of the gas-pressure type with a hollow copper conductor, pre-impregnated insulation, lead-alloy sheath, special anti-corrosive protection and armouring.

The weight of cable involved will be over 3,000 tons and special arrangements will be made for handling at both ends and shipping to site.

It will be remembered that the BICC Group collaborated with the British Electricity Authority last summer in making sea-trials for the England to France cross-channel project.

**Damodar Valley Project.** In connection with the series of articles we published recently on this scheme, we are now informed that practically all the 132 kV bulk-oil circuit breakers for the Damodar Valley project were supplied by Ferguson Pailin Limited of Manchester. In all, 33 breakers were supplied by this firm.

**Stornorrfors Plant.** In our editorial note on this plant on page 43 of our February issue we very much regret that owing to a clerical error the capacity of the three Nydqvist & Holm turbines was given as 78,000 h.p. per machine, whereas the correct figure is 178,000 h.p. As we stated in our note, the Stornorrfors plant, when completed, will be the largest in Sweden.

# Modern Fill-Type Dams

Torald Mundal, Chief Engineer, International Engineering Company, Inc., describes the factors that have to be considered with this type of structure, which is becoming increasingly popular with American engineers.

THE general public invariably thinks of a dam as a concrete structure and associates the massive exposure of masonry with the highest degree of safety, reliability, and permanency. The less impressive earth or rock-fill dams are viewed with scepticism and distrust both as to safety and durability. That exactly the opposite is true, is known only to the informed engineer who has intimate knowledge of the behaviour of dams under various conditions of loading and the influence of the elements of nature and time. The layman's mistrust for fill-type dams is sometimes hard to overcome by the engineer who may be confronted with the choice of having to increase the proportions and cost of his recommended type or adopt a less suitable structure. Rock-fill dams are particularly subject to sceptism possibly because they are of relative recent development and have a smaller number of completed structures for reference. International Engineering Company has been called upon repeatedly to elaborate on the safety of this type of dam and has carried out extensive model studies in co-operation with established specialists to investigate the behaviour of this type of dam under various loads. Much information on the safety of these structures, which is invaluable in design, has been obtained from these investigations. If the dam is properly designed by experienced engineers and construction carried out with proper skill, it can be stated categorically that all scepticism towards this type of dam is unfounded.

The art of building fill-type dams is centuries old and it is conceivable that the early dambuilders used natural barriers as a pattern. Most of these ancient structures were small, and most of them have been destroyed by the elements of nature, but many are still in use and are striking examples of the reliability and permanency of this type of dam. These structures were designed by rule-of-thumb and patterned after previously completed structures. This practice of designing, even the larger and more important dams, was continued until a couple of decades ago. At that time several things happened which contributed to an increasing demand for more basic knowledge and criteria for design of this type of dam. Several developments which occurred more or less at the same time contributed to this, the main of which are: the scarcity of suitable sites for concrete dams, the new equipment for handling and compacting fill materials in large quantities, and the new science of soil mechanics. At the present time the deficiency in basic knowledge of fill-type dams has been greatly remedied and reliable analysis of design by which the safety of these structures can be closely predetermined has been developed.

Fill-type dams have been built in preference to concrete dams to an increasing extent in later years, particularly in the United States. The type of dam which should be adopted for a certain site is largely dependent on topography, geology and location of site, and the lowest overall cost of development is the primary consideration. The most desirable dam sites, where topographic and geological conditions are most favourable, are ordinarily best suited for concrete structures and were developed first. In later years an increasing number of the less desirable sites, where foundations are poor and overburden deep, have been recommended for development. On these sites fill-type dams have definite advantages. They provide a better distribution of the resultant forces on weaker foundations and may be built on top of the overburden, saving costly excavation to solid rock. A deciding factor in favour of fill-type dams in later years has been the change in the relative cost of concrete and earth-rockfill materials, the cost of concrete having increased to a considerably higher degree than fill materials. Construction plant used in placing concrete in dams was developed to a high degree of efficiency before the war and no significant improvements have taken place in later years. On the other hand earth and rock excavating and hauling equipment has improved steadily and to such an extent that the cost of fill materials has been kept nearly constant during the later years of increasingly higher construction costs. Contributing factors are simplicity of operations and a high degree of mechanisation, eliminating manual labour to a greater extent than on most other operations.

An important consideration in the selection of type of dam is the availability of construction materials. Often this is overlooked in the early investigations and is left to be resolved later by the contractor or construction forces. What influence this may have on the cost of the project is obvious. Fill-type dams require special care in locating suitable materials in adequate quantities and within economical hauling distance of the site. Early investigations by qualified engineers cannot be overemphasised because conclusions drawn from inadequate data may necessitate radical changes during construction. Safe and dependable fill-type dams can be built on a wide variety of foundations and almost any kind of soil materials, except those with excessive amounts of organic and soluble fractions, can be used provided the dams are properly designed in accordance with these materials. Rockfill materials must be durable and of sufficient strength to withstand the superimposed load.

An adequate and qualified engineering force is required to control construction in the field on all major engineering structures. The field control of fill-type dam construction is, however, of utmost importance. The physical properties and the suitability of the materials for the different zones in the dam

must be determined before the materials are excavated and a continuous check must be made of the materials during and after placement in the dam. Design criteria usually are based on a few tests made from a limited quantity of the total material requirements. It is the field engineer's responsibility to make sure that the materials actually placed in the dam are at least equal to the materials which were assumed in the design. A close co-operation between the field engineer and the designer is therefore more important during the construction of this type of dam than concrete structures.

The three basic methods of construction serve to classify earth dams as rolled fill, hydraulic fill and semi-hydraulic fill. A number of important earth dams in years past have been built by the hydraulic fill method. Within the last few years, however, this method, as well as the semi-hydraulic method, has been selected less frequently in favour of the rolled fill. There are four factors which have contributed to this. In order to retain the fluid core the embankment must usually be designed with flatter slopes and hence requires a larger volume of embankment. Secondly, the low cost of excavating, hauling and placing rolled fill has made the hydraulic methods less competitive in most cases. Finally the range of materials suitable for hydraulic fill construction is more restricted and the material control more difficult.

Rolled-fill dams are generally classified into homogeneous fills and composite or zoned fills. A homogeneous section is constructed of essentially uniform material which is substantially watertight. In a zoned earth fill, materials of varying degree of watertightness are placed in separate parts or zones of the dam. Material most impervious to the flow of water through the dam is placed in the central portion of the section and the most pervious material forms the outside slopes. Intermediate materials are placed in zones between the two extremes. Most earth dams built in later years have been of the zoned type. If the proper materials are available, it is favoured because ordinarily it will require a minimum volume of embankment, resulting in the lowest construction cost.

Earth fill embankments must have special protection from erosive wave action within the limits of reservoir drawdown on the upstream face and within the limits of the tail water on the lower part of the downstream face. The wave height in both cases may be appreciable and adequate freeboard above maximum water level must be provided. A filter blanket between the slope protection and the fill is provided to stabilise the slope of the fill. Riprap of durable rock fragments is commonly used for slope protection. Riprap is dumped in place, special placing being unnecessary excepting that it is desirable to place the larger pieces at the face.

Rock-fill dams consist of a mass of uncemented rock fragments which provides all or the greater part of the support for the water pressure. The rock may be dumped freely or placed systematically. Watertightness is provided by impervious membranes of earth, concrete, steel, asphalt, or timber. Earth and concrete are most commonly used in present day construction. Rock-fill dams are classified into three types—inclined rock, central core, and inclined core. Local conditions at the site determine to an appreciable extent which of the three types should be selected. The inclined-deck type requires a minimum of rock-fill, but the saving in rock-fill is often more than offset by the high cost of the impervious concrete slab with its specially placed and compacted support and the high maintenance cost. This type of dam is used primarily in locations where impervious earth materials are not available. The central core and the inclined core types require little or no maintenance. The inclined core type has more advantageous construction operations and is considered the cheapest to build. The total volume of core materials required is somewhat larger than for the vertical core type, but the volume of rock-fill is slightly less.

The basic requirements for design of fill-type dams are watertightness and safety against structural failure. The two are to some extent interrelated and should be considered together. Watertightness is a relative consideration only since this type of dam cannot be made entirely watertight. Seepage must be accepted whether it occurs through the foundations or through the dam itself. The acceptable amount of leakage depends on the type of dam, site conditions, the value of the impounded water and the cost of reducing the leakage. Leakage need not impair the safety of the structure. Although forces may be present which tend to undermine the structure, these destabilising forces can be anticipated and provisions made for them in the design. The value of seepage water is rarely the criterion for determining acceptable leakage. The dam is rather made as watertight as practicable without incurring excessive cost by selecting the most suitable type of structure and the best foundation treatment. Analysis of seepage through earth dams is made by the flow net method as in the study of underseepage. The technique for embankment design is more complicated since the location of the seepage water surface within the dam must first be determined. Techniques of determining this boundary have been developed so that a seepage analysis may be made in a relatively short period of time, either graphically, by electric analogy studies, or by model tests. The principal objective of seepage control is the prevention of water emerging at the downstream face. In a homogeneous fill such a condition may be corrected by introducing an inclined pervious zone, as in Fig. 1—which gives a cross section of the Gal Oya dam. If ample quantities of both impervious and pervious materials are available, seepage is controlled effectively by proper distribution of the materials in the embankment section. A pervious section downstream from the impervious core lowers the level of the seepage line to the base of the dam, as shown in the cross section of the Arghandab Dam in Fig. 2.

The stability of the embankment slopes is commonly analysed by the circular arc method. It is sometimes convenient to use several planes rather than a circle as the potential sliding surface. The latter method is especially applicable where known planes of weakness in embankment or foundations may exist. The forces tending to move the mass are the tangential components of the weights of the moving mass. Maximum shearing resistance is determined by the forces acting on the surface and by values of cohesion and the angle of internal friction. Proper allowances are made for the pore pressure in the

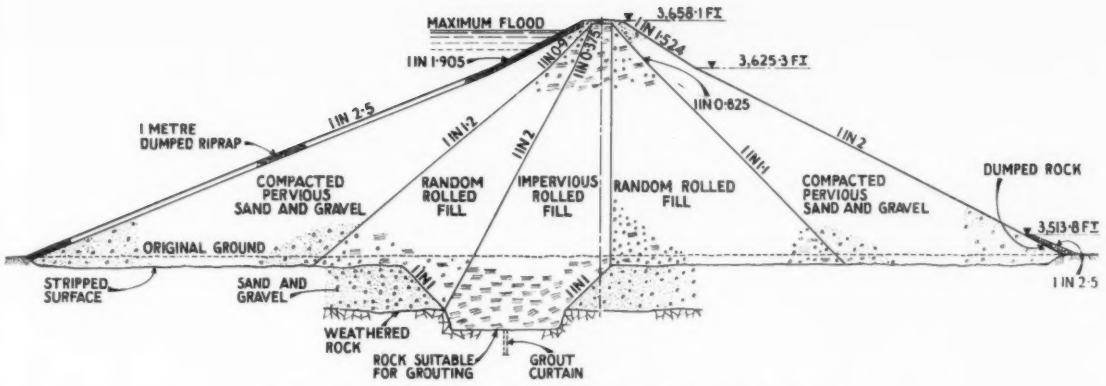



*Fig. 1. Maximum cross section Gal Oya dam, height 120 ft.*

mass. The cohesion and angle of internal friction are not constant but depend upon factors such as moisture content and density of the soil, the rate of loading and drainage. There are an infinite number of circular arcs which may be assumed as potential failure surfaces, and the most dangerous circle is determined by trial. The factor of safety of the slope is usually defined as the ratio of the maximum shearing resistance along the potential sliding surface to the tangential forces along this surface. The minimum allowable factor of safety under assumed loading conditions is a matter of judgement and depends on the method of analysis, the variability of materials, the extent of field investigations and laboratory testing, and the nature of foundations.

The embankment mass may be subject to different loading conditions and it is necessary to analyse several cases to establish the critical case. Three loading conditions are often governing. At the end of the construction period the impervious soil in the lower portions of the dam may not have consolidated and its shearing resistance may be at a low value. Secondly, a sudden lowering of the reservoir may cause sliding of the upstream face. If the fill material is relatively impervious, drainage of the embankment may be quite slow and the moving forces are increased from components of submerged soil weight to those of saturated soil weight. There is no corresponding increase in the components contributing to shearing resistance. Because of embankment impermeability, a drawdown distributed over weeks or even months may have the same effect as a sudden drawdown. The third condition is the investigation of the downstream slope for forces of water percolating through the embankment. If the dam is placed on an earth foundation, stability of the embankment and foundation is treated integrally. Values for cohesion, shear strength, and permeability used in the design of the embankment are based on laboratory test of representative samples conducted under conditions similar to those expected in the field.

Rock-fill dams may be analysed by the same method as earth-fill dams, assuming the rock-fill to be free draining and acting as a granular mass. The stability of dumped rock-fill dams against water load if fully equal to other types of dams, and the inclined-deck and inclined-core types have a factor of safety against sliding and shear far above other types.

Stability analysis of earth and rock-fill dams under earthquake loadings is complex. The forces acting on the dam and the action of the dam under these stresses are difficult to analyse. Additional horizontal forces, similar to those commonly used in masonry dam analysis, may be introduced and the outward force of the embankment may be assumed to equal a certan percentage of the sliding mass in the embankment. The validity of this assumption is questionable since the shearing resistance of a soil to momentary dynamic loading may differ considerably from its resistance to static loads. It has been established by earthquake load tests that the natural angle of repose will not be noticeably affected for values of accelerations commonly assumed in dam design. The record of earth and rock-fill dams in active seismic regions indicates that the ordinary design procedure includes an adequate factor of safety against destruction by earthquakes. The record even includes healing of earthquake displacement by plastic yielding.

In order to reduce the amount of consolidation

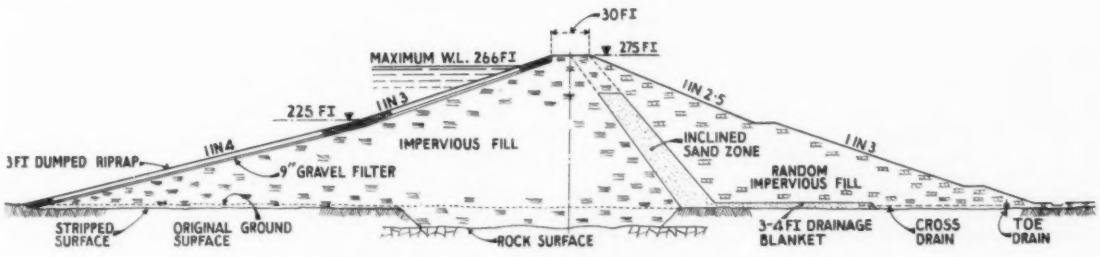



*Fig. 2. Maximum cross section Arghandab dam, height 158 ft.*

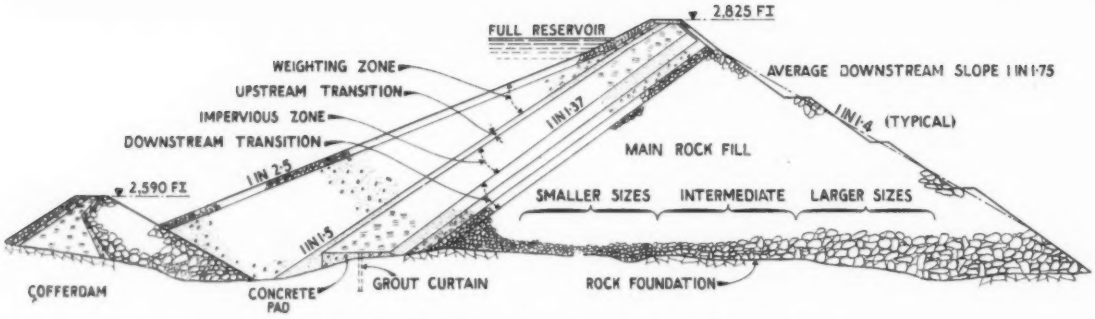



*Fig. 3. Maximum cross section, Kenney dam, height 325 ft.*

which will take place after construction, to improve shearing resistance, and to avoid piping through an earth dam, special attention is given to the compaction of the fill. Earth fills are compacted with specially designed sheepsfoot rollers on rubber-tyred equipment. The optimum percentage of moisture at which the soil may be compacted to maximum density is determined in the laboratory and the soil is placed in the fill with a moisture content close to optimum. Special test embankments are often constructed to predetermine the economical thickness of successive layers and the number of passes required by the compaction rollers to obtain the desired density. Compaction of dumped rock-fill of large fragments is improved noticeably by sluicing or washing the fill as it is being dumped with large volumes of water. This washes away the fines and assures rock-to-rock contact. Fill of small rock fragments are often compacted by specially designed vibratory equipment in relatively thin layers.

A well compacted embankment will have a relatively low percentage of consolidation after construction is completed. If the depth of the earth foundation is appreciable, considerable consolidation and settlement of the foundation may be expected. The settlement of a rock-fill dam is, however, of prime importance and must be given special consideration in the design. The vertical settlement of rock-fill is appreciable, is continued for years after construction is completed, and is roughly proportional to the height of the fill. Horizontal settlement or deformation caused by the water load is of a much smaller order.

The resultant deformation of the watertight membrane is therefore more apparent on the inclined than the vertical type. The total deformation or settlement may not be of much concern. Large differential settlement may cause cracks to develop and must be given special attention to avoid leakage and piping. An impervious earth core is less rigid than concrete and has the ability to adjust itself more readily to the settlement by plastic flow.

Proper precautions must be taken to prevent the impervious material from being washed into and through the pervious on both sides of the core. This is of special importance in rock-fill dams where the core is usually thin and the voids in the rock-fill large. Zones of filter materials are provided to stop the migration of fines and the materials are well graded from coarse to fine from the rock-fill to the core. The materials are so graded that particles in the finer zone will not enter the voids in the adjacent coarser zone; see Figs. 3 & 4 which give cross sections of the Kenney and Kajakai dams respectively.

The core is carried down to an impervious formation or to rock foundation. The extent to which the rock is jointed and fractured will generally determine the requirements of foundation grouting. To prevent excessive seepage and piping from developing along cracks in the contact zone under the core, the rock surface is often coated with gunite or concrete. This coating also serves as a grout stop enabling injection of grout under higher pressure near the surface, to fill in depressions in the rough surface, and to assure a more reliable compaction of the core material in this critical location.

Fill-type dams must not be overtopped by flood water and must be designed with adequate spillway

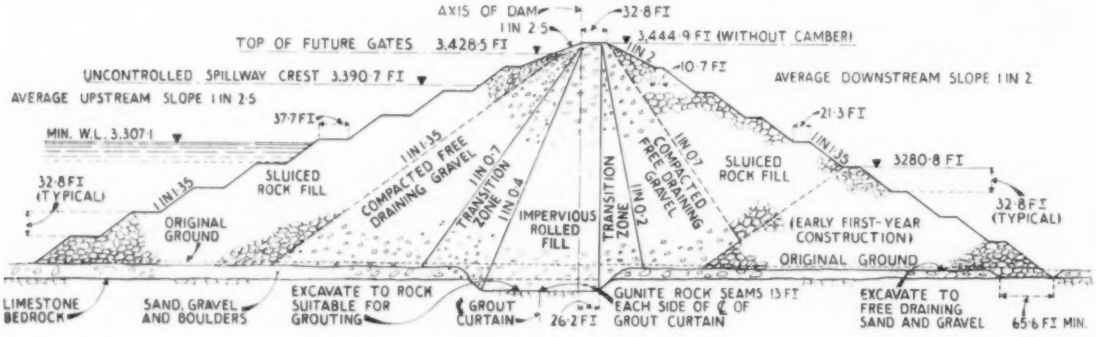



*Fig. 4. Kajakai dam, height 328 ft.*

*Fig. 5. Arghandab dam from left abutment; this dam is 1,804 ft. long and 167 ft. high, the volume of fill being 4,470,000 cu. yards*

*Fig. 6. Downstream view of Kajakai dam, Afghanistan, 885 ft. long and 295 ft. high*

capacity. Requirements for the diversion of the river during construction is usually more severe for fill type than for concrete dams, since overtopping may be too costly. The costlier spillway and diversion should be considered in comparing the relative cost.

Penstocks and conduits for water outlets are commonly located in tunnels through the abutments. If the conduits are run through the fill, special precautions must be taken in the design to prevent damage due to excessive load from the overlying fill or differential settlement. Leakage into the fill through the conduit walls must be safeguarded against and seepage along the outside conduit walls effectively prevented by providing adequate cut-offs around the conduit.

The techniques of construction and the speed with which fill materials are placed in embankments have kept pace with improvements in equipment in later years. Under favourable conditions over 100,000 cubic yards of earth materials have been excavated, hauled, placed, and compacted in a single day. The Kenney Dam, containing about 4,000,000 cubic yards of earth and rock-fill, was substantially completed in one working season between May 10 and November 27, 1952. Although operations are occasionally slowed down by wet weather and are often closed down entirely in winter time because of frost or excessive rain, fill type dams are ordinarily completed in a shorter time than other types. Earth-fill and vertical core rock-fill dams are constructed by placing materials on a working area which is substantially level across the entire section. Rock fills which require flatter slopes than the angle of repose are dumped from levels at convenient intervals. Costly rehandling is avoided by constructing the slope in steps leaving the slope at the angle of repose between dumping levels. Inclined core rock-fills are usually dumped in higher lifts. This tends to improve the consolidation of the fill and increase the speed of construction. The main supporting fill is constructed ahead of filter and core zones and without interference from other operations.

Fill-type dams built in ancient times were low structures. Recent years have seen a steady increase in both height and volume. A fill-type dam over 300 ft. high is considered rather unusual today, but with a better understanding of the forces acting on the fill and improved construction methods and control, there should be no reason to believe that a limit has been reached.

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From page 99

Ventilation is effected by a 13,000 cu. ft. per min., 22 by 60 in., Sutorbilt reversible fan. After a blast this unit is used to exhaust the dense fumes from the heading through a 24 in. diameter vent pipe for a while, after which it is put into reverse and used as a forcing fan. By this technique the air at the working face is quickly cleared and is sufficiently dispersed in the adit to enable the crew to return to load out within half an hour of the blasting.

Each foot of advance means the excavation of 21 cu. yards of rock in the solid, so that a 10 ft. round is equivalent to 210 cu. yards, or 40 to 50 car loads, as the 6 cu. yard side dumpers average slightly less than 5 cu. yards solid measure per load. A vertical lift car changer is mounted on each drill jumbo to feed the empties to a 100 h.p. Conway loader. The loaders are furnished with two electric motors, one of which handles the loading bucket and machine travel, while the other operates the belt conveyor for delivering the rock to the car. This dual motor arrangement represents a recent innovation in design which has certainly proved its value in the Kemano operations, as it enables the belt to be loaded with an almost full car load of rock in the interval between hauling out a loaded car and spotting an empty, and hence reduces the time cycle.

This cycle begins with an electric battery and trolley locomotive moving up the heading with one empty car in front and five behind. The front car is delivered to and coupled with the mucker, and while this car is being loaded the locomotive places the last car of the train under the picker by which it is lifted clear above the level of the train. The loader car is then hauled out by the locomotive to beyond the car changer, after which the suspended car is dropped on the track and pushed up to the mucker, this process being repeated until the whole train of trucks is filled.

Smooth working with this system of tunnelling demands sound training, careful timing and excellent teamwork. Months of training are required before crews can be said to be really experienced, and with many jobs of this kind the contract is not infrequently finished before full proficiency is reached. In view of this time element Kemano offers an interesting example of how improvement may continue even when it might be supposed that the crews had got fully trained. The previously recognised record for driving a 25 ft. tunnel was broken in the summer of 1952 and new records were established on three more occasions before the end of the year. Unfortunately, after the Christmas recess, many of the trained miners did not return to the job and it took six weeks before the crews got into their old stride. In the week ending February 21, 1953, however, one crew advanced no less than 282 ft. in six days, a record advance of 61 ft. being made in one day. If it be accepted that an advance of 10 ft. per shift is a reasonable rate of progress, this average of 47 ft. per day for a week represents an outstanding achievement. Taking all four headings together, an advance of 125 to 135 ft. per day is achieved.

Reference to the diagram reproduced in Fig. 14 will show that at the 2,600 ft. level the main tunnel branches out into two penstocks which first incline at an angle of 48 deg. from the horizontal, then follow a more or less horizontal course for 685 ft. and then renew their 48 deg. slope until they reach the powerhouse level at an elevation of 200 ft., at which point they level off to join the valve chamber. Taken together, the two penstocks represent 2,797 ft. of horizontal driving and 6,412 ft. of inclined driving. Access is gained through the powerhouse, through a 1,170 ft. long adit at the 1,600 ft. elevation and through a 450 ft. tunnel at the top. Drilling and blasting were accomplished on much the same lines as those used for the main tunnel, excepting that at the powerhouse level a truck-mounted jumbo was used to carry the drilling equipment and the loading was effected by a 1½ cu. yard Eimco overhead tractor loader; also diesel trucks were used to haul out the muck.

*(To be continued)*

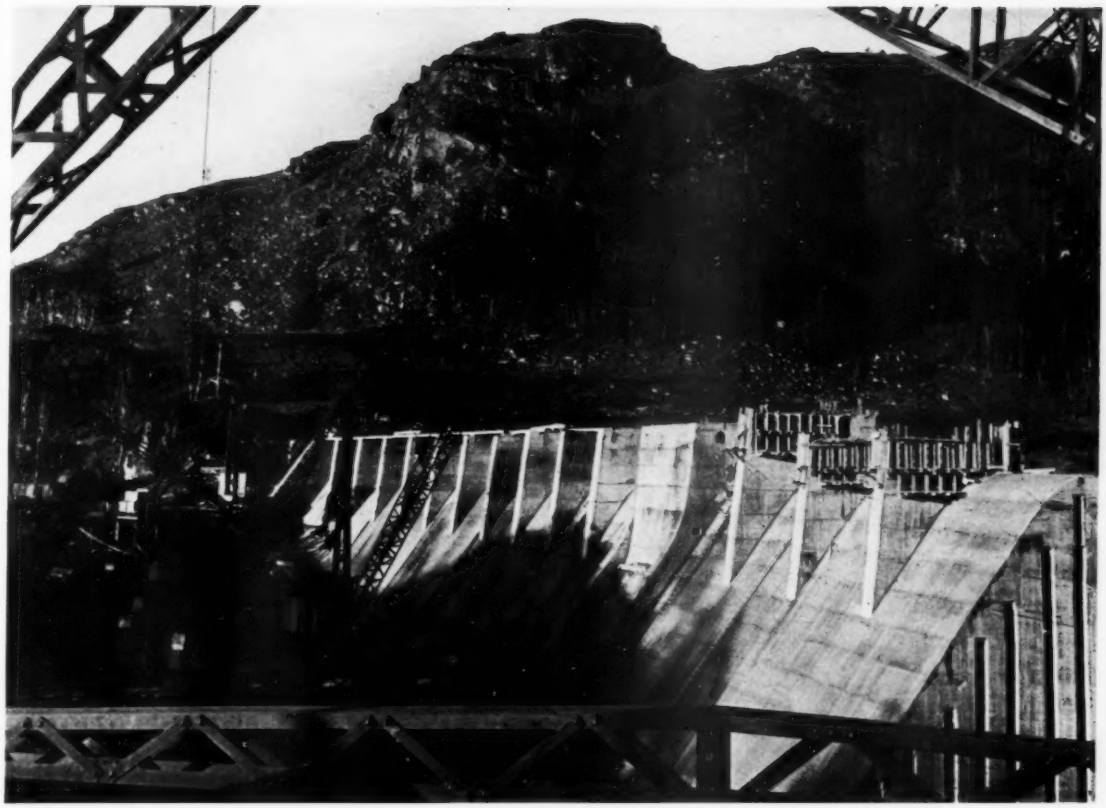

*Fig. 1. A view of the Luichart dam under construction*

# Conon Valley

Some particulars are given of this scheme which will constitute the largest hydro-electric project in Britain.

ON December 22, the North of Scotland Hydro-Electric Board invited a party of journalists to inspect the progress made with this Ross-shire scheme, the first items in which were begun in 1946 and the last are scheduled for completion in 1959. Ultimately it will consist of six power stations with an installed capacity of 107,000 kW and an annual output of 442 million kWh. Our issues of October and November 1951 gave an outline of this scheme and detailed descriptions of the first completed items—the Grudie Bridge power station and the Loch Fannich tunnel. This last is 3½ miles long and was connected to the loch by "Operation Bathplug," which consisted of the blowing out of a plug of 600 tons of rock between the tunnel and the bottom of the loch where it is rising towards its east end.

To appreciate the size and complexity of the scheme, and the remarkable number of times the same water is used over again, it is necessary to study the map. The whole area from which water will be drawn is about 25 by 25 miles and the main development known as the Glascarnoch-Luichart-Torr Achilty scheme will use water from 345 square miles and produce 290 million units per annum.

The scheme centres on the river Conon, which, flowing out of Loch Luichart, enters the sea near Dingwall; above Loch Luichart it is known as the river Bran. The Conon has two utilisable southern tributaries, the Orrin and the Meig, and one northern one, the Blackwater. The Blackwater in turn has two northern tributaries, the Rannoch and the Vaich, and is known in its upper reaches as the Glascarnoch river. Work started at Fannich and on the Bran-Conon line, but the scheme can best be followed from the north. At Altguish Inn on the Glascarnoch and at Strathvaich Lodge on the Vaich, dams will be built of earth and rockfill construction, to save cement and reduce the need of shuttering, and will form the artificial lochs of Vaich and Glascarnoch. In the extreme north Glen Beag will be tapped by tunnel and

aqueduct to feed the Vaich loch, and the Rannoch is also being connected to it by a tunnel. Another tunnel will take these waters to the Glascarnoch reservoir and from here a five-mile tunnel, now under construction, will take the combined waters to Glascarnoch power station on Loch Luichart.

The Loch Fannich and Grudie Bridge parts of the scheme have already been described and it need only be added that a dam is projected on Loch Fannich at some future date. On the Bran, a short distance above Grudie Bridge, a small barrage and power station are to be built, called Achanalt after the loch of that name, so that in all three power stations will discharge into Loch Luichart. At the foot of Loch Luichart is the Luichart dam. At Curin on the Meig a dam is being built to create the artificial Meig loch and from it a tunnel has been driven to bring the water to Loch Luichart above the dam, whence a tunnel, also completed, will take the water to Luichart power station near the junction of the Meig and the Conon. Finally a dam and power station are being built on the Conon opposite a hill called Torr Achilty, thus creating another artificial loch.

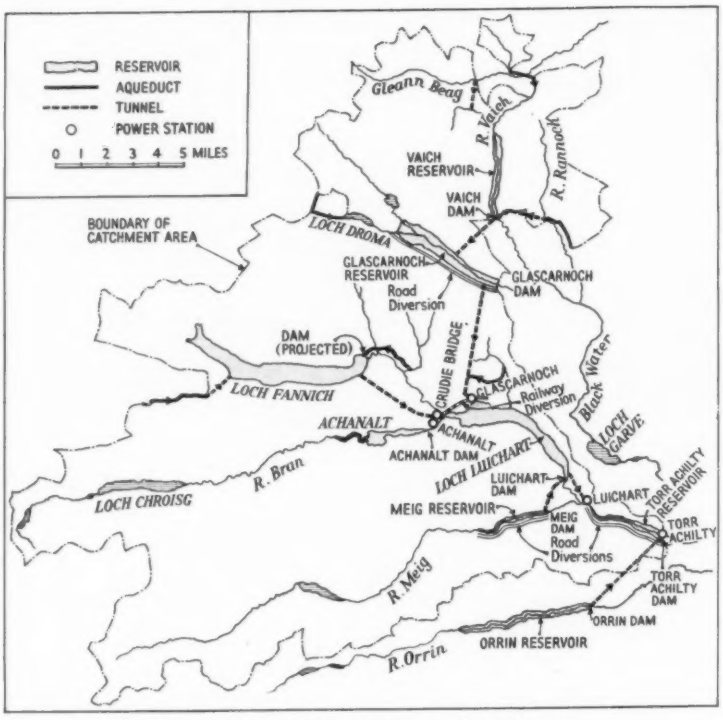


*Fig. 2. Map of the Conon Valley scheme*

It is hoped shortly to obtain sanction for a third stage of the general scheme to include a dam and artificial loch on the Orrin from which water will be taken by tunnel to a power station on the south side of Torr Achilty loch at Achonokie. The annual output of this scheme is estimated at 74 million kWh and the water discharged from the Orrin power station will pass through the Torr Achilty turbines and a further 5 million kWh will be gained in this way.

The features of the various items in the scheme are summarised in the table below.

The contractors are:—

Reed & Mallik, Salisbury, for Luichart, Glascarnoch, and Vaich dams, Rannoch aqueduct, Glascarnoch road diversion and Achanalt civil works.

William Tawse Ltd., for Torr Achilty dam and power station.

A. M. Carmichael Ltd., for Glascarnoch and Vaich tunnels.

George Wimpey & Co., for Luichart tunnel.

Duncan Logan, Dingwall, for Meig dam and Luichart power station.

J. Campbell & Son, Inverness, for Glascarnoch power station.

M.A.P.C.O. for Loch Luichart railway diversion embankment.

Thomas W. Ward for track of railway diversion.

P. & W. McLellan, Glasgow, for new welded railway bridge, Loch Luichart, and Luichart pipeline.

Glascarnoch is equipped with Harland turbines and Bruce Peebles generators, Luichart with English Electric turbines and generators, and Torr Achilty with Boving turbines and Metropolitan-Vickers generators. The overhead cranes for the same stations are by Marshall, Fleming, William Arrol, and Babcock & Wilcox.

The consulting engineers for the scheme are Sir Alexander Gibb & Partners.

About 1,400 men are at present employed on the Glascarnoch-Luichart-Torr Achilty scheme, on tunnels, dams, power stations, road diversions, etc. The party began its inspection at Torr Achilty dam, which

| | Fannich | Glascarnoch | Achanalt | Luichart | Torr Achilty |
|---|---|---|---|---|---|
| Main tunnel: | | | | | |
| Length ... ... ... | 18,480 ft. | 23,011 ft. | 542 ft. | 4,090 ft. | — |
| Diameter ... ... | 11 ft. | 10 ft. | 11 ft. | 16 ft. | — |
| Plant capacity ... ... | Two 12,000 kW | As Fannich | One 2,000 kW | As Fannich | Two 7,500 kW |
| Gross head ... ... | 530 ft. | 529 ft. | 65 ft. | 185 ft. | 52 ft. |
| Estimated average output: million kWh ... | 79 | 112 | 6 | 124 | 36 |

| | | | |
|---|---|---|---|
| Glascarnoch dam ... | 1,690 × 90 ft. | Meig dam ... | 565 × 55 ft. |
| Vaich dam ... ... | 780 × 85 ft. | Meig tunnel ... | 8,930 × 12·9 ft. |
| Vaich tunnel ... | 11,498 × 9 ft. | Torr Achilty dam ... | 570 × 60 ft. |
| Luichart dam ... | 680 × 52 ft. | | |

is interesting for two reasons, the first being that its function will be to even out the flow of water downstream in much the same way as the Pitlochry dam operates on the Tummel-Garry scheme. For that reason it has only a 2 ft. range of level and is fitted with flood gates. Secondly, it will have a Borland fish lift to enable salmon to get upstream. This is worked on the same principle as a canal lock; a gate is closed at the foot of a shaft and the salmon floated up to the higher level. The power station, which will form part of the dam structure, will be entirely of stone obtained from a quarry near Tain.

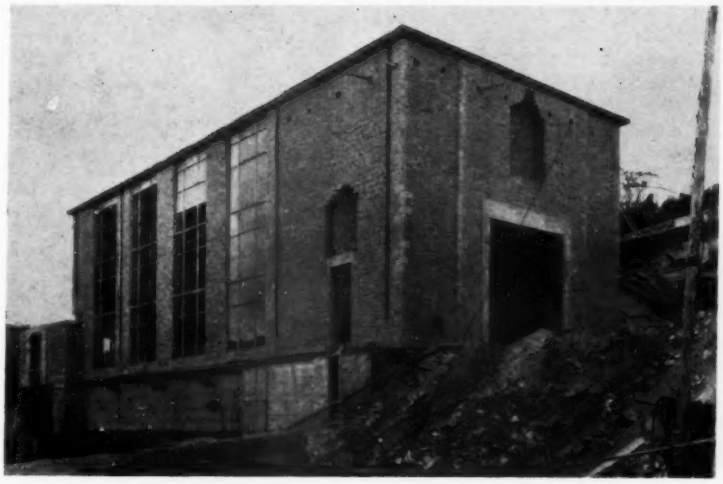

*Fig. 3. Luichart power station in course of construction*

The Luichart dam, having a 20 ft. range of level has spillways over the top and is to have two Borland fish locks. The Board is under an obligation to avoid damaging the fishing attractions of the district, and in this scheme has actually added to these attractions. Half a mile below Luichart dam are the Conon Falls—three falls of ten, twenty, and thirty feet respectively. Ten feet is about the limit of a salmon's leap, so that from time immemorial no salmon have been able to pass these Falls. The Board will build the usual step-type fish pass at the Falls, and this, combined with the requisite fish locks higher up, will enable salmon to get right through to Achnasheen. In this way, 18 miles of new salmon fishing will be created, and to help in the good work the Board have planted 200,000 salmon fry in the Upper Bran and will continue to add to the stock yearly.

In connection with the Luichart dam the Meig tunnel has been completed and is being lined; the tunnel to the power station is also complete. This new station will have nine times the power of an old

*Fig. 4. Another view of Luichart dam taken from the east*

Grampian Power Company station built in 1928 near the dam—a station that is still extant but now out of use.

Luichart power station was the first to be built entirely of stone instead of brick or concrete with stone facing. A beautiful rose pink sandstone obtained from Tarradale quarry near Muir of Ord has been used and three masons and six apprentices are busy completing the structure. A grey mica schist is being used for embankments, etc.

Later the party was taken by Contin and Garve to Grudie Bridge (passing the fish hatchery at Contin and fish trap at Loch-na-Croic), not so much to see the power station as to note the railway diversion necessitated by the projected raising of the level of Loch Luichart. The existing Loch Luichart railway station and nearly two miles of railway will be submerged, so a complete new station and railway embankment and track has had to be built, and a new single-span all-welded railway bridge, 100 ft. long, erected over the Bran; it was prefabricated in Glasgow and placed in position in one day.

*Fig. 5. Strathconon road diversion*

Summarising the position, Mr. Thomas Lawrie, C.B.E., the Board's General Manager, stated that, of the five tributaries of the Conon, one had been put to work, three were being harnessed, and it was hoped that sanction for the fifth, the Orrin, would be obtained very shortly.

Sir Hugh Mackenzie, deputy chairman of the Board, later gave a general account of the Board's progress, remarking that it was responsible for an area of 21,750 square miles or about three-quarters of the whole area of Scotland. Their problem was that, as time went on, it became increasingly difficult to find schemes that were economical by reason of the scattered nature of the country to be served; they took it as a strong indication of the people's keenness to obtain electric power that in small and scattered communities in Sutherland they could get as many as 65 people to come to a meeting to discuss it. In 1948 in the whole northern area there were only 9,107 consumers connected; at the end of November 1953 there were 17,593, that is, the number had nearly doubled in five years. The total number of potential consumers in the northern area is 21,700, so that 81 per cent. have been connected, leaving 4,000 still to connect. Dealing with the same theme, Mr. Lawrie stated that at the start of the Board's activities only one croft in a hundred and one farm in fourteen were connected; now the figure for both farms and crofts was one in three and it was hoped to connect the remainder in the next decade.

Sir Hugh said the latest proposal was a big distribution scheme to supply Strathspey, Strathnairn, and on to Foyers, at a cost of about £350,000, and he gave some interesting figures for road diversions. In Inverness-shire the Board have built or are building 53 miles of road at a cost of £560,000, and in Ross-shire 18 miles at a cost of £300,000. The Board are merely required to "replace" submerged roads but it was generally acknowledged that they not only replaced but improved. On the subject of fish he referred to the trap and hatchery above mentioned and stated the Board had employed three men to catch predatory fish in Lochs Luichart, Chuilinn, and Achanalt. In two seasons 29,000 perch and 3,750 pike had been destroyed.

The Board are very proud of their successful endeavour to keep the art of the stonemason alive. In addition to the four power stations which were to be built of stone, Tarradale stone was also being used to build a control station at Fort Augustus and at Loch Quoich in Glengarry. In all 45 masons, including 12 apprentices, are distributed over the Board's works.

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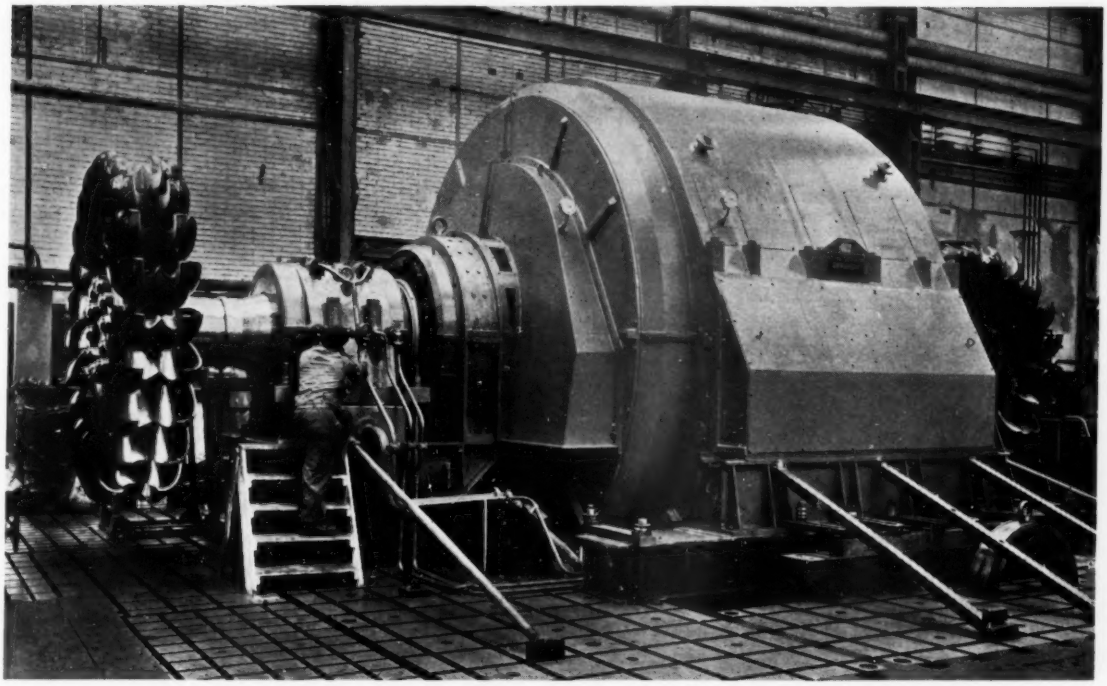
**Bitumastic Pipeline Coatings.** Wailes Dove Bitumastic Limited have issued a data book relating to their protective coatings for pipelines. An introductory section discusses the causes of corrosion—concentration cell effect, galvanic cell effect, stray-current corrosion, bacterial effect, etc.—while a further section deals with the properties that a good pipeline coating should possess. Succeeding sections discuss hot enamelling and wrapping, cold applications, testing, and give data relating to various types of pipelines and the coatings recommended in each case.

# Montpezat Power Plant



*One of the 63,000 kVA Escher-Wyss / Oerlikon generating sets for Montpezat*

The Montpezat power scheme, now approaching completion, utilises the waters of the upper basin of the river Loire and exhibits some rather original features. This plant represents the first example in France where streams normally flowing to one sea are diverted into another, i.e. water flowing hitherto towards the Atlantic Ocean will flow into the Mediterranean Sea. The basic idea of the project was to capture waters from slopes with considerable rainfall and to utilise them in a concentrated fall.

The power house, which is built underground, has two power units, each with two bearings and a horizontal shaft turning at 428 r.p.m. As shown in the picture, there is a Pelton runner, manufactured by Escher Wyss Limited, mounted on each side of an Oerlikon generator. Each wheel is driven by two jets and the two together develop 81,250 h.p. under a net head of 625 m. The centrally situated generator is designed for 63,000 kVA 0·93 power factor at 15,500 V. It is self-ventilated, the same air moving in a closed circuit through the generator and the water coolers.

To reduce the length of each unit and thereby the cost of the underground constructional work, the main exciter is located inside the bearings as shown in the picture. However, the auxiliary exciter, the permanent-magnet pendulum generator and a small generator for auxiliaries constitute a group which is connected to one end of the main shaft by a flexible coupling. As these machines are rather small for this speed it was decided to have them mechanically coupled in preference to other possible arrangements.

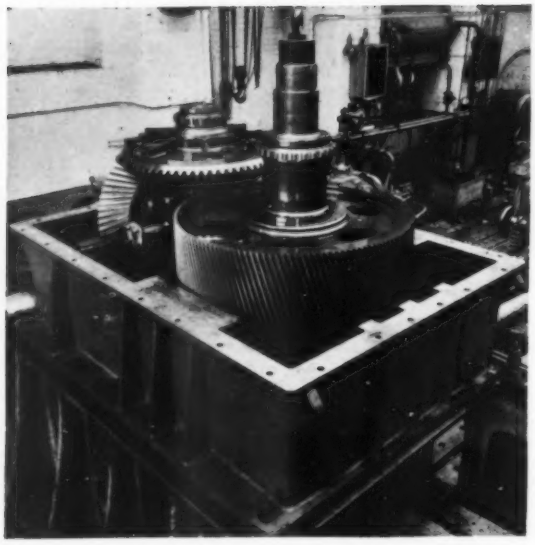
The efficiency of the generator at full load is 98·21 per cent. at unity power factor and 97·98 per cent. at 0·93 power factor. These measured values were within 0·01 per cent. of the guaranteed figures. The total weight is 248 metric tons. The general construction of the generator follows that usually adopted by the Oerlikon Engineering Company for units of comparable size and speed.

## Turbine Gear for Mullardoch

In our description of the Glen Affric scheme (WATER POWER, May, June and July 1952) we referred to a subsidiary generating plant which was to be installed to make use of a head of about 80 ft. between Loch Affric and Loch Benevean. Loch Affric forms the main storage for the scheme, and supplies Loch Benevean through a tunnel 5,738 yards long and 15 ft. 9 in. equivalent diameter, the combined waters being taken through a further tunnel to Fasnakyle power station.

Adjacent to the Loch Affric end of the Affric-Benevean tunnel an underground turbine house has been constructed to accommodate a Gilbert Gilkes & Gordon vertical Francis turbine. The turbine spindle is carried up the vertical access shaft to drive a 2,400 kW horizontal Bruce Peebles induction generator in a surface building in which all mechanical and electrical controls are located.

This layout involves a right-angled gear drive, and the accompanying illustration shows this drive, which has recently been completed, in the works of David



*David Brown 3,600 h.p. turbine gear*

Brown & Sons (Huddersfield) Limited. It is a speed-increasing gear, the vertical input shaft rotating at 208 r.p.m. and the horizontal output shaft at 755 r.p.m., and comprises a single-helical spur gear followed by a spiral-bevel gear. Below the spur wheel is a Michell thrust bearing, and elsewhere ball and roller bearings are fitted throughout. A forced lubrication system is incorporated. The drive has a continuous rating of 3,600 h.p.

## Cajurú Storage Dam

This gravity-type dam has a volume of 50,000 cu. m. of concrete and a total length of 438 m., of which 341 m. are of concrete and 97 m. of earth. It is located on the Pará river and will permit the operation of the Gafanhoto hydro-electric plant at full capacity (18,600 h.p.).

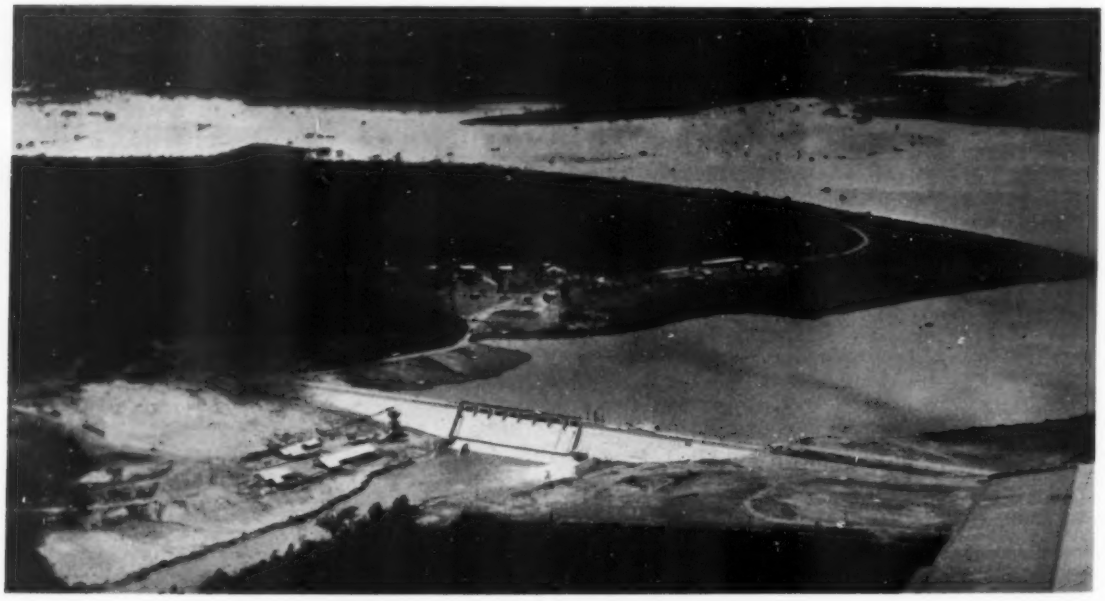
The reservoir shown partly filled has a gross storage capacity of 170 million cu. m. (net 161,500,000 cu. m.) and an area of 2,300 hectares. Discharge control is effected by two 2·44 m. diameter Howell Bunger type valves, each with a capacity of 30 cu. m. per sec. These works are owned by Centrais Elétricas de Minas Gerais S.A. (CEMIG) who entrusted the constructional work to Cia. Morrison Knudsen do Brazil S.A.

## Amplidyne Exciters for Owen Falls Generating Sets

The first two 16,700 kVA alternators for the Owen Falls hydro-electric scheme are being supplied by The British Thomson-Houston Co. Ltd. and are being fitted with an excitation system that differs from that hitherto generally used on alternators of this type in that it incorporates amplidyne control.

In place of the conventional main and pilot exciters, a d.c. generator is mounted on the top of the alternator shaft, which supplies both the alternator field and the driving motor of an auxiliary exciter set comprising an auxiliary exciter and an amplidyne. Voltage and current references are taken from the alternator output and referred to an electronic voltage regulator This regulator acts on the field of the amplidyne, from which the auxiliary exciter receives its excitation. The auxiliary exciter bucks or boosts the alternator field as may be required. Sliprings are provided on the d.c. generator to give an a.c. supply to the voltage regulator, which is thus independent of all outside supplies, and to the pendulum motor of the turbine governor.

The advantage of this method of excitation is that the range of excitation and the speed of the response



*An aerial view of Cajurú storage dam*

are increased. Main and pilot exciters on the alternator shaft are largely due to the low speed and inherently have a slower speed of response than the auxiliary exciter and the amplidyne operating at 1,000 r.p.m. As the auxiliary exciter operates directly on the alternator field, and is therefore independent of the response ratio of the d.c. generator, the overall rate of response is much improved. An additional advantage, which may be very important in some installations, is that the total height of the alternator and perhaps the crane and station height, if these are not fixed by other considerations, are reduced.

# Abstracts from the World Technical Press

## Venezuelan Hydro

The Venezuelan Ministry of Development has approved the vast Caroni River Falls hydro-electric project. Situated in the State of Bolivar, the falls are part of the region where the Orinoco Mining Company, a U.S. Steel subsidiary, has dredged a river, built a harbour and a city, and run a railway and a highway to Carro Bolivar, the "Iron Mountain." *(Engineering News-Record*, Vol. 151, No. 20, 12.11.53, p. 63.)

## Sutherland Dam Nears Completion

Work on the 1,240 ft. buttress-and-arch dam on Santa Ysobel Creek, east of San Diego, was resumed in July, 1952, after a lapse of 25 years caused by lack of funds as well as disputes over riparian rights of land owners. Very little design change was made in the original plans of this 17-arch structure; the struts between the buttresses were, however, modified for earthquake protection. Buttresses, spaced at 60 ft. centres, range in thickness from 10 ft. at the bottom to 3 ft. 4 in. at the top; arches are 6 ft. at the base and 2ft. at the crest, which is 161 ft. above the stream bed. Foundations, extended to solid rock, vary between 5 and 60 ft. Completion of the project, originally scheduled for December, 1953, had now been postponed to March, 1954. *(Engineering News-Record*, Vol. 151, No. 20, 12.11.53, p. 48, 2 pp., 2 ff.)

## Energy Production in Turkey

This general survey gives some interesting information on hydro-electric development in Turkey. The country is not rich in energy resources but those available, as far as is known, are more than sufficient to cover the present demand because of the relatively slow progress of industrial development. Compared with the area of Turkey, hydro-electric production, assessed at 15,000 million kWh, is rather small, and is restricted to coastal regions. The most important hydrographic basins are those of the Sakaria and Kizil Irmak rivers, which flow into the Black Sea; the Gediz and Menderes which flow into the Ægean Sea; the Mediterranean Seyhun and Ceyhan; the Aras, in Turkish Armenia; and the Tigris and Euphrates, which have, however, a most irregular regime. While thermo-electric output grew from 17,300 kW in 1913 to 456,000 kW (1,014 million kWh) in 1952, hydro-electric production began as late as 1940 with a modest 11,000 kW and did not exceed 33,000 kW (51 million kWh) in 1952.

In addition to small thermo-electric plants, two classes of hydro-electric plants are to be constructed in conjunction with the 1952/56 Five Year Plan:—

(a) Medium-size plants, varying in capacity from 4,000 to 7,000 kW each and involving a total expenditure of T£10 million.

(b) Large plants, comprised in an extensive programme of water-supply regulation, involving together an expenditure of 100 to T£200 million. Among these major installations a 80,000 kW plant is nearing completion at Sariyar, west of Istanbul, and other plants are under construction on the rivers Seyhun, Gediz and Böyük Menderes. These plants will supply the cities of North-West Anatolia—Ankara, Zonguldak and Izmir—in addition to Constantinople, and it is expected that they will be able to cover the demands until 1960.

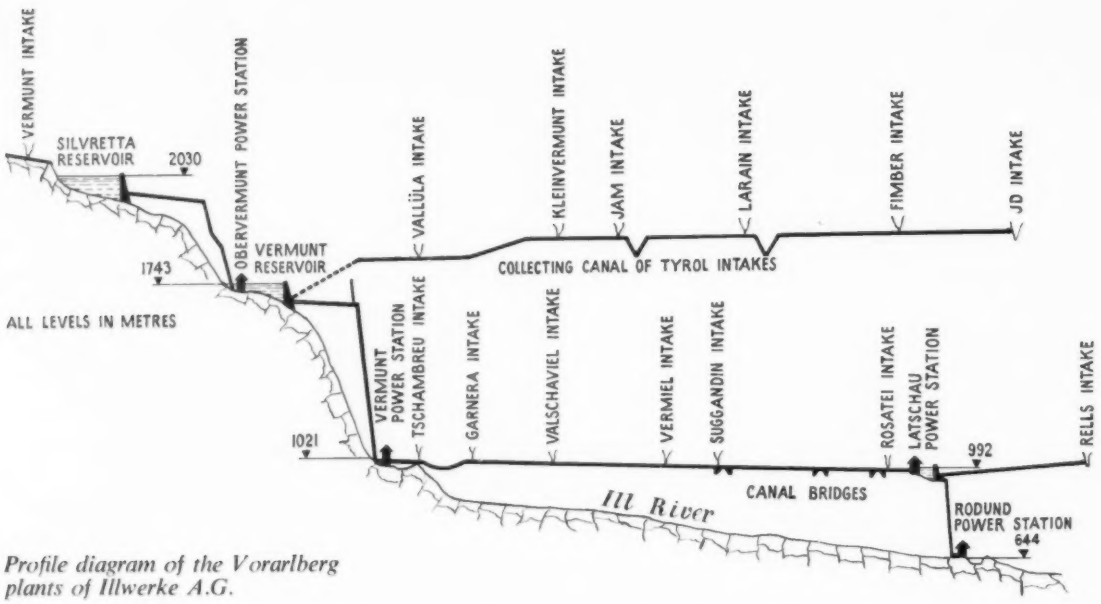
In spite of a comparatively rapid increase in production, the total number of consumers of electric energy did not exceed 409,000 at the beginning of 1949, out of a total population of nearly 20,000,000, and no more than 231 towns and villages were connected to supply systems. This situation had not experienced any change worth mentioning by 1952, when the aggregate production of all electric undertakings was assessed at about 967 million kWh, and the yearly consumption per head of the population has only increased from 37 kWh in 1950 to 46 kWh in 1952. (*Quaderni di studi e notizie*, Vol. 9, No. 146, 1.5.1953, p. 312, 3 pp., 3 tables.)

## New Edison Group Plants

A general survey is given of the new plants added to the Edison Group since 1949 in the basins of the Toce, Liro-Mera and Oglio rivers, as well as in the Trentino Province. Also reviewed are the plants of subsidiary companies, thermal plants, transformer stations, and 220 kV transmission lines. A complete map of the plants and other installations is appended to the article. (*Quaderni di Studi e Notizie*, Vol. 9, No. 151, p. 513, 5 pp., 2ff.)

## Gal Oya Valley Development

After a hydrological survey of the Gal Oya catchment area, a description is given of this multi-purpose project, initiated by the Government of Ceylon with a view to protecting a cultivated area from floods, supplying hydro-electric power, improving the distribution and control of water and, above all, providing for better irrigation. The rolled-fill Gal Oya dam is about 154 ft. high by about 272 ft. long, the crest being at an elevation of 272 ft. The earthwork involved was about 5·5 million cu. yards. The concrete spillway structure is 1,000 ft. in overall length and 65 ft. high, the spillway proper being 770 ft. long. Six 5·75 x 5·75 ft. sluices are provided. The dam, on which work started in March, 1949, was completed in November, 1951. A sluice and power house are included in the headworks. The water of the 770,000 acre-ft. reservoir will be used for power generation before being made available for irrigation; as power demand increases, two further units will be added to the two existing units. The possibility of establishing




*Profile diagram of the Vorarlberg plants of Illwerke A.G.*

a sugar-cane industry that would draw power from the plant is to be investigated. Aggregate useful power resources are assessed at 12,000 kW. (Dr. T. Mylvaganam, Senior Research Officer, Ceylon, *Indian Journal of Power and River Valley Development*, Vol. III, No. 8, August 1953, p. 6, 5 pp., 5 ff.)

## Vorarlberg Hydro-Electric Plants

A brief description is given of the plants of the Illwerke A.G., Bregenz, visited by a section of the Conference of the *Studiengesellschaft für Höchstspannungsanlagen* (Research Association for extra-HV plants), which was held in Constance.

The Illwerke A.G. is an Austrian concern, financed to the extent of 89·1 per cent. by two German companies operating mostly thermal plants, together with the Württemberg Provincial Government. Their Vorarlberg plants aggregate an installed generator capacity of 330 MW, plus two 35,000 kW storage pumps, of which but one is in service; they operate during the morning and afternoon hours and, conversely, draw the energy stored by the pumping plant during the night and lunch-interval hours, which correspond to the off-peak load in the German factories supplied. The exchange of energy between the Ill works and the German system thus reverses direction four times a day. As shown in the appended diagram, there are at present four power stations in operation.

Obervermunt, the uppermost plant (1,734 m.), draws its water from the Silvretto reservoir (2,030 m.), close to the watershed separating the Danube-Black Sea basin from the Rhine-North Sea basin; this 38·6 million cu. m. storage feeds the 15,000 kW Francis turbines of the two 19,000 kVA sets of this fully automatic plant, remotely controlled from the Vermunt power house. In addition to the discharge from the Obervermunt turbines, the Vermunt reservoir also collects water from various subsidiary intakes in Tyrol. The Vermunt power station, which operates against a head of 720 m., houses a 39,000 kVA and four 30,000 kVA alternators, all driven by single Pelton wheels. The Vermunt discharge collects in a 110,000 cu. m. basin and then flows through a channel about 18 km downstream to the Latschau reservoir. To utilise the discharge and head available at this point (the latter varying from maximum to minimum between 28 and 11 m.) a smaller intermediate power house has been erected at Latschau, with two 5,000 kVA generators driven by Kaplan turbines which discharge into an artificial lake constituting the forebay of the last step of the system, the Rodund plant, about 450 m. below. Of the four 45,000 kW horizontal Francis turbines driving 53,000 kVA alternators at Rodund, two are coupled each to a 52,000 h.p. pump; at present, one of these pumps only is in regular operation twice a day, driven by the corresponding alternator which, acting as a motor, absorbs the energy produced at night and during the midday interval. Before returning to the Ill, the Rodund discharge is collected in a reservoir, to be pumped back, when required, into the Latschau storage.

This four-step Ill scheme, however, is but a beginning; the erection of numerous further plants is contemplated downstream of Rodund, and it is also planned to harness to storage—for an additional power station—the Lünersee, a natural lake situated in a valley lateral to the Ill on its left bank. *(Quaderni di Studi e Notizie*, Vol. 9, No. 155, p. 637, 3 pp., 2 ff.)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